A TEXT-BOOK

ON

ROOFS AND BRIDGES.

PART III.

BRIDGE DESIGN.

BY

MANSFIELD MERRIMAN,

PROFESSOR OF CIVIL ENGINEERING IN LEHIGH UNIVERSITY,

AND

HENRY S. JACOBY,

PROFESSOR OF BRIDGE ENGINEERING IN CORNELL UNIVERSITY.

FOURTH EDITION, REWRITTEN.

FOURTH THOUSAND.

NEW YORK:

JOHN WILEY & SONS.

LONDON: CHAPMAN & HALL, LIMITED.

1905.

First edition, September, 1894; reprinted, 1895.
Second edition, August, 1896.
Third edition, February, 1898; reprinted, 1899.
Fourth edition, rewritten, April, 1902; reprinted, 1903, 1904, 1905.

Set up and electrotyped by J. S. Cushing & Co., Norwood, Mass.
Printed and bound by Braunworth & Co., Brooklyn, N.Y.

PREFACE.

THE present edition has been entirely rewritten in order to bring the subject fully up to date, for the changes in bridge design during the past eight years have been remarkable. The rapid increase in live loads on the principal railroads in this country has necessitated an unusually large amount of new construction to replace the old bridges, which were designed for much lighter traffic. The extensive scale on which this work had to be done led to a general revision of specifications and to careful attention to the design of details so as to secure greater stiffness as well as strength in the new structures. These changes include the introduction of some new forms of details; the elimination, as far as practicable, of adjustable members; the entire superseding of wrought iron by steel; the substitution of riveted trusses for pin-connected trusses in the shorter spans; and increased care in designing the joints so as to reduce the secondary stresses to a minimum.

In the descriptions of the details of plate-girder and truss bridges, introduced in Chapters VI, VIII, and XI, only those are given which may be properly claimed as standard in the best recent practice. Special attention is called to the new feature of carefully selected references to engineering periodicals where more extended descriptions and applications of various details may be found.

The designs in Chapters VII, IX, and X are new, being made in accordance with the latest specifications and most approved practice. As stated in the preface to the first edition, the subject is presented "both rationally, as an application of the principles of mechanics, and practically, as an illustration

of modern economic construction. Since probably more than ninety percent of all bridges are those resting on two supports, this volume is confined to that class. For a beginner the study of bridge design should be largely that of proportioning details according to given specifications, and simple bridges furnish these in endless variety."

Grateful acknowledgments are due to many railroad and bridge engineers for kind assistance: to RALPH MODJESKI and E. H. MCHENRY for permission to reproduce three sheets of standard plans; to J. A. L. WADDELL for permission to reprint the larger portion of his specifications for steel railroad bridges for simple spans; to C. C. SCHNEIDER, J. E. GREINER, W. J. WILGUS, W. A. PRATT, F. W. SKINNER, and A. F. ROBINSON for photographs and drawings; to ENGINEERING NEWS and ENGINEERING RECORD for permission to reprint those illustrations which are marked with their respective names; to THADDEUS MERRIMAN for the chapter on Bridge Shops and Shop Practice; and to F. O. DUFOUR for the chapter on the Design and Detailing of a Highway Bridge.

A comparison with the third edition shows that the number of pages has been increased from 316 to 374, and the number of cuts from 57 to 149, of which 20 are full-page illustrations; the number of folding plates is the same, but all of these are new. In rewriting the volume, it has been the constant aim of the authors not only to give the latest details of modern bridge practice, but also to set forth the reasons for such practice in a manner especially adapted to the needs of students and young engineers.

MARCH, 1902.

CONTENTS.

FOLDING PLATES.

FULL-PAGE ILLUSTRATIONS.

BRIDGE DESIGN.

CHAPTER I.

HISTORY AND LITERATURE.

ART. 1. EVOLUTION OF GIRDER BRIDGES.

All bridge structures may be divided into three classes, Beam Bridges, Suspension Bridges, and Arch Bridges. Beam bridges exert only vertical pressures upon the abutments or piers, suspension bridges exert a horizontal pull, and arch bridges exert a horizontal push in addition to the vertical pressures. Beam bridges include simple bridges, drawbridges, continuous bridges, and cantilever bridges. A simple bridge is one resting upon two supports; and probably over ninety percent of all bridges are of this kind. Parts I, II, and III of this work are devoted entirely to simple bridges, while the other forms are discussed in Part IV.

Simple bridges are of two classes, girder bridges and truss bridges. A truss bridge is one whose floor is supported by two or more frameworks, called trusses, each consisting of two chords, which are connected by bracing. A girder bridge, on the other hand, has its floor supported by solid or built-up beams. A wooden beam, a rolled I-beam, and a plate girder, formed by riveting angles and plates together, are examples of girders. Girder bridges are used for short spans, usually less

than 100 feet, while truss bridges are used for longer spans, and sometimes for spans as short as 50 feet.

Probably the first bridge was merely a tree-trunk that had fallen over a brook; later, several trees or logs placed side by side, and covered perhaps with brush and earth, formed a structure of greater convenience for the traffic of semi-civilized people. When the stream was too wide for a single span, a rude pier of piles or stones was built to support the ends of logs extending from it to the shore. Several piers of this kind were built for still wider streams, and thus arose the trestle structure, in which each span consisted of simple beams. The oldest wooden bridge on record, the famous "Pons sublicius," built across the Tiber, at Rome, about 650 B.C., is believed to have been of this kind, as also was the bridge built by CÆSAR over the Rhine in 55 B.C.

Little progress beyond the simple wooden beam was made until the early part of the nineteenth century, when cast-iron beams began to come into use. It was then soon recognized that the depth of the beam was a controlling factor in its strength, and that the greatest economy of material resulted by forming the cross-section so that the upper and lower parts should be thicker than the middle part. Thus arose the flanges and the web of a girder, the flanges carrying the greater part of the horizontal stress, while the web served mainly to hold the flanges together. Such cast-iron beams, with ⊥, ⊔, and [ ] sections, were used before 1840 for bridges on many English railroads, the longest span of a beam cast in one piece being 46 feet. These bridges, however, proved unsatisfactory on account of the low tensile strength and unreliability of the metal.

The first wrought-iron rolled beams were made in England about 1820 for railroad rails, and their use for other purposes slowly increased in both Europe and America, so that, by 1875,

I-beams up to 15 inches in depth were obtainable. Since 1890 medium steel has rapidly replaced wrought iron, so that now all I-beams are rolled of this material, and sizes up to 20 inches in depth and 30 feet in length are readily found in the market. Many deck bridges of 30 feet span or less have been built with such beams, and they also are extensively used for the floors of buildings and bridges.

About 1850 built-up plate girders, formed by riveting angles to a solid web plate, began to be used in Europe, and later were

Fig. 1.

introduced into this country, where they are now extensively employed for spans ranging from 30 to 100 feet. Fig. 1, from a photograph, shows the plate-girder bridge of 80 feet span, built in 1892 at Ithaca, N. Y., on the Delaware, Lackawanna and Western Railroad. The largest spans of plate-girder bridges are from 120 to 140 feet in length, the depth of the girders being about 12 feet. They are stiffer than truss bridges, and the shorter spans have advantages in erection, as a girder may

be made entire in the shop and swung into place by a derrick, the only field-riveting required being that necessary to connect the girders by lateral bracing.

A tubular bridge is a girder structure with its sides formed of plates and stiffeners, and its top of channels, angles, and plates, all being riveted together so as to form a closed tube. This type originated in England about 1840, and in 1850 STEPHENSON built the great Britannia bridge in Wales on this plan, the tube being 25½ feet high and 13¾ feet wide inside, and there being four spans, two of 230 feet and two of 460 feet. The Victoria bridge over the St. Lawrence River at Montreal, completed in 1859, was of this type, but it was replaced in 1898 by a truss structure. These tubular bridges, though stiff, were unnecessarily heavy, and accordingly very expensive, and the passage through them was like that through a tunnel. All experience indicates that the girder system of construction cannot be economically applied to bridges of long span.

A lattice truss, or lattice girder, as it is sometimes called, consists of flanges formed like that of the plate girder, but with the solid web replaced by flat, diagonal bars. The Warren truss, with a double system of web bracing (Part I, Art. 64), originated in England about 1840, and it may be regarded as being an attempt to economize material by removing unnecessary parts of the web. This was a step in the right direction, as the web stresses were thereby more closely determinate than before. But, as will be seen in the following articles, greater precision regarding stresses and greater economy in material have been attained by discarding the double set of diagonals, and using only a single system of bracing to connect the chords.

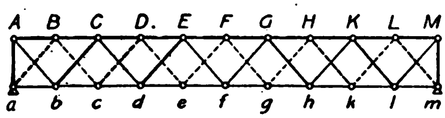


Fig. 2.

ART. 2. TRUSS DESIGN PRIOR TO 1800.

Bridge design prior to the year 1800, and indeed for many years after, was almost wholly empirical. Arch bridges of stone had been successfully built since the time of the Romans, and structures of timber were used for roofs and often for bridges, but the true idea of a bridge truss and of the functions of its members was not fully understood until near the middle of the nineteenth century. About 1830, owing to the introduction and development of railroads in both Europe and America, bridge construction assumed an importance never before known. In Europe the main line of evolution was based upon the metal girder, as described in the last Article. In America, however, the evolution was along the line of the truss, starting with timber and gradually developing into structures of iron and steel. A truss is a framework whose members are so arranged that they are subject only to longitudinal stresses of tension and compression. These members should be arranged in triangular figures so that no distortion of the structure can occur without bringing the proper stresses into action, and the applied loads should preferably be concentrated at the joints (Part I, Art. 23). The simple truss, supported at its two ends, is the one whose history is now to be considered.

The king-post truss shown at *a* in Fig. 3 may be supposed to be the origin of all modern bridge trusses. Prior to 1800,

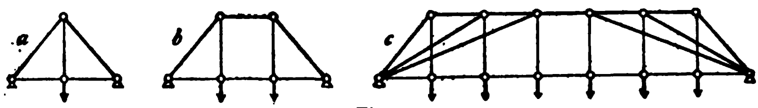


Fig. 3.

however, the principal line of development was that indicated by the diagrams *b* and *c*. On this plan many wooden bridges were erected during the seventeenth and eighteenth centuries. There were two chords, usually with a high camber, connected

by vertical timbers acting as ties to support the floor which was placed along the lower chord. From the top of each vertical an inclined brace was carried to the nearest abutment and the tops of the corresponding pairs connected by a straining beam. True truss action as we now understand it scarcely existed, the main idea being to carry each load to the abutment by the shortest route. This was a simple plan, but it proved uneconomical on account of the long braces whose stresses increase both with their length and the angle of inclination to the vertical. On this plan was built, in 1760, by GRUBENMANN, a timber bridge near Baden, which had the great span of 366 feet, and which exhibited much skill in carpentry.

The secret of economical and efficient truss arrangement lies in the panel system, which may be regarded as having been

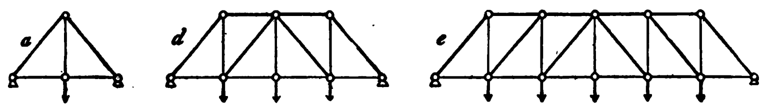


Fig. 4.

developed from the king-post truss in the manner shown in Fig. 4, where *d* is derived from *a* by the addition of a panel on each side, and *e* from *d* in like manner. This system was first used by PALLADIO, an Italian architect, about 1570, but it produced little or no influence on methods of construction, until it was rediscovered and used in the United States near the close of the eighteenth century by THEODORE BURR. The Burr truss may indeed be called the parent of nearly all the forms of bridge trusses now used in this country. Although so defective from the lack of counterbraces that it generally required the assistance of an arch to stiffen it under rolling loads, yet as it contained the sound principle of economy in a constant angle for the inclined members its panel system was transmitted to the Long truss, the Howe truss, and later to many other forms (Part I, Art. 25).

Concerning early timber bridges, as also for other valuable historical and descriptive matter, the student should consult COOPER's American Railroad Bridges, 1890, the article Bridge in the Encyclopædia Britannica, and the article Bridges in JOHNSON's Universal Cyclopædia, 1897.

ART. 3. PROGRESS FROM 1800 TO 1850.

Near the beginning of the nineteenth century many wooden bridges were erected in the eastern and middle states by THEODORE BURR and by TIMOTHY PALMER, both of whom used the panel system. PALMER's bridges generally combined the action of the truss and the arch in one structure, the lower chord being highly cambered, while BURR used the arch merely as auxiliary to the truss. The oldest truss bridge now standing in the United States is that built by BURR at Waterford, N. Y., in 1804, which is of hewn yellow pine, having four spans of 154, 161, 176, and 180 feet in the clear. Illustrations of this bridge and of one built by PALMER at Easton, Pa., in 1805 are given in COOPER's American Railroad Bridges. WERNWAG's great bridge of 340 feet span, built at Philadelphia in 1812, also deserves notice as a splendid example of early engineering work; the double diagonal bracing in its panels showing that probably its builder had considered the distorting action of rolling loads.

The lattice truss introduced by TOWN about 1820 consisted of planks pinned together, and was important only on account of ease of construction. In 1830, however, a radical advance was made in the true principles of truss arrangement through the introduction of panel counterbraces by S. H. LONG. In a pamphlet published by him in 1836 the function of counterbraces in preventing the distortion of the panels under rolling loads, and also their use in stiffening the truss when keyed up

so as to be under initial stress, is clearly recognized. Wooden Long trusses were erected on the Baltimore and Ohio Railroad as well as many for highway service.

In 1840 WILLIAM HOWE patented a combination truss having wooden chords and web diagonals and wrought-iron vertical ties, which has since been extensively used. Each panel had counter as well as main struts, both butting against cast-iron angle blocks. Many important bridges were built on this plan prior to 1850, the most notable being that over the Susquehanna

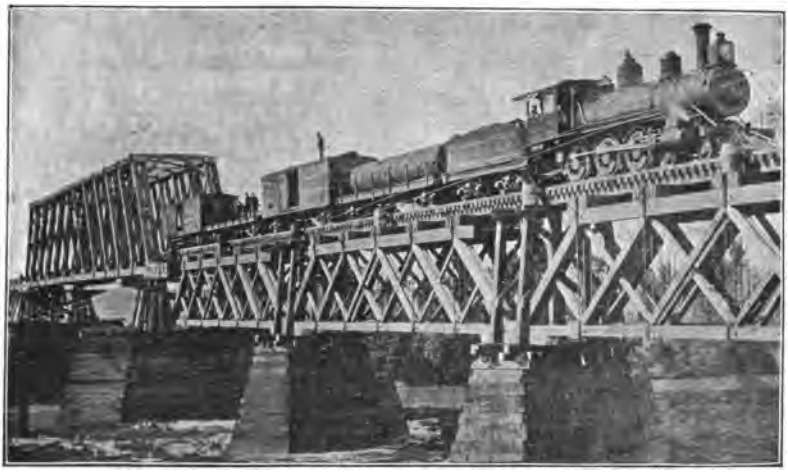


Fig. 5.

at Havre de Grace, Md., which had thirteen spans of 250 feet each and a draw span of shorter length. The Howe truss is still in common use in localities where timber is cheap, and for short spans and light traffic it often makes an efficient and economical bridge. Fig. 5 shows a Howe truss bridge of several spans over the Stanislaus River, near Riverbank, Cal., and on the Atchison, Topeka, and Santa Fé Railway.

In 1844 the Pratt truss was introduced. In this a radical departure was made in the arrangement of the web members,

Fig. 6. Baltimore and Ohio Railroad Bridge crossing South Branch of Potomac River in West Virginia.

the timber verticals being made to take compression, and the iron diagonals to take tension. This was a move in the direction of economy, since the length of the struts was decreased and thus the necessary cross-section somewhat diminished. Although at first built as a combination bridge, it never attained great popularity in this form, but soon after 1850 it began to be constructed entirely in iron, and in this form it has probably been more extensively used than any other form of truss. Fig. 6 shows a Pratt truss bridge of two spans, each $172\frac{1}{2}$ feet long, erected in 1901.

Few iron structures were built in the United States prior to 1850, the first one being a span of 77 feet erected in 1840 over the Erie Canal, which was formed of cast-iron girders strengthened by wrought-iron rods. About the same time WHIPPLE built a truss with a curved upper chord of cast iron and a straight lower chord of wrought iron, forming the bowstring truss. A Howe truss in iron was introduced in 1844, and the Rider iron truss with a multiple web system was first built about 1847, but neither came into general use, and some that were built failed.

The first rational discussion of the determination of stresses and proportioning of cross-sections of truss members was published in 1847 at Utica, N. Y., by SQUIRE WHIPPLE under the title A Work on Bridge Building, in which are given methods of computing stresses due to dead and live loads, investigations as to the angle of economy for web bracing, with plans and details of the bowstring truss and of the double system Pratt truss, since known as the Whipple truss. WHIPPLE was far in advance of his time in rational views of economic truss design, but the circulation of his book was small, so that its influence was not fully exerted until several years after publication. He also built over twenty bridges on his plans which gave good service for many years. SQUIRE WHIPPLE is justly regarded

as the father of American rational bridge design. Drawings of bridges built between 1840 and 1850 may be seen in DUGGAN'S Stone, Iron, and Wood Bridges of United States Railroads, 1850; and also in HAUPT'S General Theory of Bridge Construction, 1851.

ART. 4. TRUSS EVOLUTION SINCE 1850.

The modern bridge truss is the result of an evolution or development in the sense that it exhibits those features which experience has found to be most economical and stable. Forms costly or unsafe have disappeared, while those cheap and strong have remained in use. Thus, the panel system has survived, while the method of transferring loads directly to the abutments by long braces, as seen in Fig. 3, has gone out of use. The Bollman truss, introduced soon after 1850, was an instance of the application of that erroneous principle, but it could not be built for spans greater than 160 feet, and even for shorter spans it was unable to compete in economy and stability with trusses of the panel system. The Fink truss (Part I, Art. 53) is another example of the use of that principle, and it too has disappeared.

The Whipple truss (Fig. 7) is an instructive instance of a form which was extensively used from 1850 to 1885, even for the longest spans, but which now is no longer built. This has

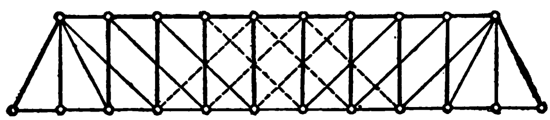


Fig. 7.

all the advantages of the Pratt type as regards the use of vertical compression members in the web, and also by the double system of webbing the panel points are brought nearer together, thus decreasing the length of the stringers, which for long

spans is a matter of importance. Stringers longer than 25 feet make an expensive floor; and this limits the economic depth of the Pratt truss to about 30 feet, and the span to about 300 feet, since it is not advisable to make the depth less than one tenth of the span. With the Whipple truss, however, using the same angle for the bracing, the depth of the truss can be doubled, and the span thus be economically increased. Many long bridges have been erected on this plan, among which may be mentioned the 515-feet span of the Ohio River bridge at Cincinnati, completed in 1877, and which at that date was the longest truss span ever erected. The Whipple truss began to go out of use merely because it was found to be more economical to support the floor beams by short sub-verticals attached to a single system of bracing than by the use of a double system, and because the single system is always more reliable and determinate in respect to stresses. The Post truss (Part I, Art. 55) is another example of a form once popular and now no longer in use.

The Warren or triangular truss was built with both single and double systems of webbing, but with a single system it afforded opportunity for the support of intermediate floor beams in a panel by the use of independent vertical members. In 1869 the channel span of 390 feet over the Ohio at Louisville was built on this plan, and in 1885 the 525-feet span at Henderson. This plan has been found advantageous because simplicity of truss action is secured, the only apparent disadvantage being the use of long inclined compression members in the webbing; in accordance with the law of evolution the former of these tends to be perpetuated and the latter to disappear.

At the present time the Pratt truss is most generally used for short spans. The Baltimore truss (Fig. 8) is used for both short and long spans; it possesses all the advantages of the Pratt, and in addition that of supporting intermediate floor beams by the use

of sub-verticals. The modified bowstring truss, shown in Fig. 9, uses the same idea, and here is gained the important advantage that the stresses in the chords are rendered closely uniform, as also those in the webbing. These elements combined

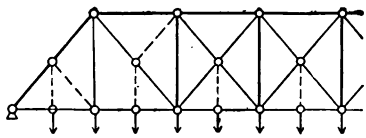


Fig. 8.

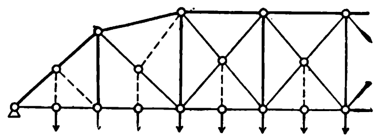


Fig. 9.

have rendered this form applicable to the longest simple trusses, the longest of all being in the great spans of 542½ feet built over the Ohio at Cincinnati in 1888 and of 546½ feet at Louisville in 1893. Fig. 10 shows one span, 533 feet long, of the Delaware river bridge of the Pennsylvania Railroad, built in 1896.

To recapitulate, the principles which should control the arrangement of a simple truss are the following: first, the panel system whereby the inclined members in the webbing are kept

Fig. 10.

at approximately the same angle; second, the use of counterbraces to prevent distortion under a rolling load; third, that compression members should be made as short as possible; fourth, that a single system of webbing is always preferable,

and that intermediate floor beams may be supported when necessary by the use of independent verticals; and fifth, that the form of the truss should be such that the stresses in members of the same kind may be approximately equal.

In addition to the references at the close of Arts. 2 and 3 the following may be noted as treating of the development of trusses: Bridge Superstructure, a committee report in Transactions of American Society of Civil Engineers, 1878, Vol. 7, pp. 339–368; an Address by JOSEPH M. WILSON in Proceedings of the Engineers' Club of Philadelphia for 1889, Vol. 7, pp. 65–104; The Evolution of the Modern Bridge by CHARLES D. JAMESON in Popular Science Monthly, Feb. 1890, pp. 461–481; and the Evolution of Bridge Trusses by MANSFIELD MERRIMAN in Railway Age for May 19, 1893, Vol. 18, pp. 381–393.

ART. 5. MATERIALS USED IN BRIDGES.

Prior to 1840 wood was the material used in this country for bridge construction. Great skill in carpentry was developed to devise the joints, mortises, keys, and other connections, although little was known regarding the strength of the timber or the rational principles of designing the proportions of the parts. The Howe truss combined the use of wood and iron in a most simple and successful manner, wrought-iron adjustable tie rods being used for the vertical members of the web, while the wooden diagonals butted against cast-iron angle blocks. In the original Pratt truss, cast-iron joint connections were also employed, through which the wrought-iron diagonal ties passed. The first bridges wholly in iron had the compression members of cast iron and the tension members of wrought iron, this being, as WHIPPLE advocated, the best theoretic combination, since cast iron is high in compressive and low in tensile strength. Wrought iron, more-

over, was high in price, and could then scarcely be obtained except in the form of simple rods.

Bridges of cast and wrought iron were built extensively until about 1870, and many of short span since that year; but the numerous failures of the cast-iron parts led to the gradual substitution of wrought iron. Probably the first bridge in which both compression and tension members were made of wrought iron was that erected on the Lehigh Valley Railroad at Mauch Chunk in 1863, but in this cast-iron joint connections were used. Gradually but surely wrought iron displaced cast iron, both for truss members and for joint details, so that by 1875 cast iron was regarded as a material wholly inappropriate for use in bridge structures for railroad purposes, and the period of wrought-iron bridge development was at its height. But about this time steel began to be introduced.

The first extensive application of steel was in 1873 in the arches of the great St. Louis bridge, and later it was used in the trusses of the Brooklyn suspension bridge. In ordinary trusses it was at first employed in the form of eye-bars for tension members, and then for the webs of floor beams. But improvements in the methods of manufacture soon followed, so that by 1890 angles, beams, channels, and other shapes of medium or mild steel were easily obtainable in the market at the same price as those of wrought iron. This structural steel closely resembles wrought iron, but its strength is about ten or fifteen percent higher, and hence in 1900 it had entirely replaced wrought iron in bridge construction.

The average life of iron or steel railroad bridges is probably not far from twenty years, although under heavy traffic many are replaced after fewer years of service. They are liable to corrode from atmospheric influences and from the gases from the locomotives, while rivets and other connections are worn

and loosened under the frequently repeated stresses and shocks. Bridges built twenty years ago are now generally unable to carry the heavier rolling stock with a proper margin of security. Hence a metallic structure cannot compete with stone with respect to durability, and accordingly many roads are replacing short spans by arches of stone. The cheapness of iron and steel, however, generally renders metallic structures more economical in spite of their shorter life, and of course for long spans no other materials are available.

Some interesting notes by SQUIRE WHIPPLE on early iron bridges will be found in Railroad Gazette, April 19, 1891. A historical paper on steel manufacture in America by W. F. DURFEE is given in Popular Science Monthly, Oct. 1891, pp. 729–749. See also COOPER's American Railroad Bridges, originally published in Transactions American Society of Civil Engineers for 1889, Vol. 21, pp. 1–28.

ART. 6. JOINT CONNECTIONS.

The members of the early wooden bridges, such as the Burr truss and the Long truss, were connected together by means of joints devised especially for timber structures. The fish and scarf joints employed in the chords are still used in the Howe truss and in other wooden constructions, but most of the special devices of shoulders, mortises, and keys now exist only in a few isolated examples.

The combination trusses which next followed, like the Howe and Pratt, employed the method of screw connections to join the webbing to the chords. In the Howe truss the several pieces of the chords were bolted together laterally, and connected longitudinally by fish joints so as to form one continuous member, but the web struts butted against angle blocks and were held in place by screwing up the vertical iron tie rods.

The Pratt truss in its early forms had wooden chords upon which was placed at each panel point a cast-iron joint block, and through this passed the diagonal iron ties which terminated in screws and nuts by which the whole was held in place. This method was also extensively used in the Pratt trusses built of cast and wrought iron, and many special forms of screw connections were devised and employed. In general, however, most of these screw joints have gone out of use, on account of the greater cheapness and reliability of the methods of riveted and pin connections.

The riveted system of connections is the prevailing method of construction in Europe, but in this country it is mostly limited to plate girders and to lattice trusses less than 200 feet in span. In this system the chords are formed of angles, or channels, and plates, riveted together, with splice joints so as to make them practically continuous from end to end; and the web members are connected to the chords by rivets, either directly or by means of special plates riveted to both. The first riveted bridges in this country were erected on the New York Central Railroad about 1860, and the system has proved very serviceable there and elsewhere.

The pin system of connections is the one which has been most used and which has generally been regarded with the most favor by American engineers. At each panel point a pin, or round bar, passes through holes in the chord or web members and serves to transfer the longitudinal stresses from one member to another by means of the shearing and bending stresses generated in it. Some of the early bridges built by WHIPPLE had pins which passed through looped eyes in the tension members, but the first bridge which was pin-connected throughout was erected by J. W. MURPHY in 1859 on the Lehigh Valley Railroad at Phillipsburg, N. J. Wide forged eye-bars in connection with pins were first used in 1861 by

J. H. LINVILLE on the Pennsylvania Railroad. The system then rapidly spread on account of ease of erection, and thousands of pin-connected bridges are now in service.

Much might be said in comparison of the riveted and pin systems. Advocates of the former claim that it makes a stiffer structure and one less liable to accident from the failure of a single member. Advocates of the latter say that the stresses in the pin system are more determinate and that better workmanship is secured. But under present conditions the question of economy seems the controlling factor. A long span cannot be built as cheaply by the riveted system as by the other, and a short or medium span can sometimes be built more cheaply. Under proper specifications a good bridge can be designed and erected on either plan, and the item of cost will usually determine the decision. The riveted system generally requires a little more material than the pin system, and the latter requires more skilled workmanship. High prices for iron and labor were favorable to the development of the pin system, and as these become lower the riveted system comes more and more into use. The literature noted in the preceding articles contains much information regarding the various methods of joint connections. Further reference is made to the works named in the following pages, and also to a series of articles on Expired Bridge Patents by F. B. BROCK, in Engineering News during 1882 and 1883.

ART. 7. LITERATURE OF BRIDGE DESIGN.

The computation of stresses in the principal members of a bridge truss is the least part of the work of design, and hence books treating mainly on stresses are not noted in the following list. Bridge design includes of course the economic principles regarding the form of the truss, some of which have been mentioned in Art. 4, but more specifically it is the science of

details, that is, the proportioning of the members, the floor, the joint, and of all the splices, reënforcing plates, rivets, pins, and other parts which make up the structure. The list of books below includes such as treat wholly or in part of these topics, together with a few of historical and descriptive character. Although not complete, it is believed that it gives the works on Bridge Design most important for a college library and for the use of American students of bridge design. The list is arranged chronologically according to the date of the first editions.

POPE, T. A Treatise on Bridge Architecture. New York, 1811. This contains 196 pages of descriptions of early bridges, while the remainder is devoted to the author's "patent flying pendant lever bridge."

WHIPPLE, S. A Work on Bridge Building. Utica, N. Y., 1847, pp. 120 and 10 plates. The edition of 1869 contains also 128 pages of notes (printed by the author's own hands) explanatory of the original work. See Art. 2.

DUGGAN, G. Stone, Iron, and Wood Bridges of United States Railroads. New York, 1850. Consists mostly of drawings, with brief descriptive notes.

HAUPT, H. General Theory of Bridge Construction. New York, 1851, pp. 268 with 16 plates, giving examples of railroad bridges.

VOSE, G. L. Handbook of Railroad Construction. Boston, 1857, pp. 480. Contains 109 pages on wood, iron, and stone bridges.

HUMBER, W. Cast and Wrought Iron Bridge Construction. London, 1864, two volumes, with 80 plates, mostly descriptive of English bridges.

HEINZERLING, F. Die Brücken in Eisen. Leipzig, 1870, pp. 515. A historical and descriptive work on bridge develop-

ment in all countries. Also Die Brücken der Gegenwart. Leipzig, 1884, pp. 754 with 60 plates.

MERRILL, W. E. Iron Truss Bridges for Railroads. New York, 1870, pp. 130. A comparison of seven kinds of trusses with respect to theoretic economy.

BOLLER, A. P. Construction of Iron Highway Bridges. New York, 1876, pp. 144. Although written for the use of town committees, this book has been of much value to young engineering students.

DU BOIS, A. J. Strains in Framed Structures. New York, 1883, pp. 390 with 27 plates. This devotes 124 pages to design, and gives the complete design of a pin-connected bridge. The edition of 1896 has 209 pages on design and erection.

WADDELL, J. A. L. Designing of Ordinary Iron Highway Bridges. New York, 1884, pp. 244 and 7 plates. A book which has done much to improve the design of highway structures.

BENDER, C. Principles of Economy in the Design of Metallic Bridges. New York, 1885, pp. 195 with 9 plates. This does not treat of details, but gives critical theoretic comparisons of different forms of trusses.

RICKER, N. C. Construction of Trussed Roofs. New York, 1885, pp. 158. Mainly deals with stresses, but has two chapters on dimensions and details.

BURR, W. H. Stresses in Bridge and Roof Trusses. New York, 1886, pp. 454 with 12 plates. Devotes 112 pages to details and to the design of a railway bridge.

SCHÄFFER, T., and SONNE, E. Der Brückenbau (Vol. II of Handbuch der Ingenieur Wissenschaften). Leipzig, 1886–90, pp. 1812 with 77 plates.

HIROI, I. Plate Girder Construction. New York, 1888, pp. 94. Gives the design and estimate for a span of 50 feet.

MORANDIÈRE, R. Traité de la Construction des Ponts et Viaducs. Paris, 1888, pp. 1891, with 332 large plates.

COOPER, T. American Railroad Bridges. New York, 1890, pp. 58 with 27 plates. A historical and descriptive work of special value.

FOSTER, W. C. Treatise on Wooden Trestle Bridges. New York, 1891, pp. 160 with 38 plates. Gives many standard plans, accompanied by their bills of material.

JOHNSON, BRYAN and TURNEAURE. Modern Framed Structures. New York, 1893, pp. 517 with 37 plates. This gives 238 pages on details, with designs of several bridge structures.

WARREN, W. H. Engineering Construction in Iron, Steel, and Timber. New York, 1894, pp. 372 with 13 plates. Devotes 92 pages to the details and designs of simple span bridges, besides the designs of several other classes of bridges.

WRIGHT and WING. A Manual of Bridge Drafting. Stanford University, 1896, pp. 214 with 51 plates and 5 blue prints. Gives tables of shears and moments for girders, and details for different types of bridges.

BERG, W. G. American Railway Bridges and Buildings. Chicago, 1898, pp. 705. Gives many illustrations of details of timber structures, and other information compiled from reports of railroad superintendents.

WADDELL, J. A. L. De Pontibus: A Pocket-Book for Bridge Engineers. New York, 1898, pp. 403. Gives general specifications, and many tables and diagrams for facilitating computations.

A number of monographs on large bridges have also been issued in book form, which are of special value to advanced students and engineers. Among these are The Quincy Bridge, by T. C. CLARKE, 1869; The Kansas City Bridge, by O. CHANUTE, 1870; The Omaha Bridge, The Cairo Bridge, The Bellefon-

taine Bridge, and others, by G. S. MORISON, 1889–93; and The Thames River Bridge, by A. P. BOLLER, 1891.

The Transactions of the American Society of Civil Engineers contain many papers both descriptive and critical. Of the latter class may be noted 'Specifications for the Strength of Iron Bridges,' by JOSEPH M. WILSON, in 1886, Vol. 15, pp. 410–490; 'Some Disputed Points in Railway Bridge Designing,' by J. A. L. WADDELL, in 1892, Vol. 26, pp. 77–282; and 'The Launhardt Formula and Railroad Bridge Specifications,' by H. B. SEAMAN, in 1899, Vol. 41, pp. 140–268. The volumes of Engineering News, Railroad Gazette, Engineering Record, and other technical periodicals, contain numerous articles, both theoretical and descriptive, on bridge design, and some of these will be mentioned in the following chapters. The Index of Engineering Literature, published by the Association of Engineering Societies, in 1892, and by the Engineering Magazine, in 1896 and 1902, gives many pages of titles of such articles, with brief notes of their contents; and this should be at the hand of every student who desires to become well informed on the progress of bridge development. But it cannot be too strongly urged upon the student to form the habit of making his own catalogue of articles, and of giving under each title his own synopsis of its contents and conclusions. By so doing he acquires a training in technical literary work which will be of the greatest value in promoting his professional advancement.

CHAPTER II.

PRINCIPLES OF ECONOMIC DESIGN.

ART. 8. DATA OF THE DESIGN.

In order that the most economic design may be made for a bridge it is necessary that complete data regarding its location should be known. An accurate map of the locality, showing the neighboring roads or streets, should be prepared, as also a profile of the crossing, giving the high and low water marks of the stream and the character of the earth or rock below its bed. This profile should be extended some distance from each bank of the stream in order to enable the approaches of the bridges to be properly arranged. The location of the bridge and of its abutments and piers are to be shown on the map, while the grade line of the bridge and its approaches are given on the profile. If there are more spans than one, the position of the piers is determined by making approximate estimates of their cost in different positions and then applying the principles of Art. 9.

In locating the abutments and piers it is always advisable to avoid a skew, as thereby the cost of the superstructure will be increased. When this cannot be done, as in the case of one street crossing another obliquely or in the case of a stream with rapid current, the angle of skew should be made as small as possible and the same in amount at each end of a span. In locating the grade line of the floor of the bridge the clear waterway desired is to be considered, as also the grades of the approaches; these will also determine whether the bridge is

to be a through or a deck structure or whether certain spans should be through and others deck.

Facts regarding the regimen of the stream, such as its velocity at both low and high water, its liability to freshets at different seasons of the year, and the amount of drift carried during freshets, are useful to a bridge company in estimating the cost of erection. The distance from the bridge site to the nearest railroad siding should also be stated in order that estimates of the cost of cartage may be made. The loads to be carried by the bridge, the lateral clearance required between trusses, and the vertical clearance needed for through bridges must be carefully specified. The kind of floor desired, the width and number of sidewalks, if any, should be stated. With these facts on hand the engineer is ready to prepare a general plan for both substructure and superstructure and to write specifications from which the detailed designs may be prepared. Time spent in gathering data is always usefully employed, for experience has shown that most of the mistakes and losses that have occurred in bridge construction have been due to imperfect knowledge of the local conditions.

ART. 9. NUMBER OF PIERS AND SPANS.

When a bridge is to be built across a river, one of the first considerations is that regarding the number of spans. This question is to be decided by the principle that the total cost of the substructure and superstructure shall be a minimum. In any event there will be two land abutments; and if the distance between these be short, no intermediate piers are advisable. Yet it is seen even here that if piers could be erected without any expense, it would be best to use them. Thus the relative cost of piers and their connecting spans determines the number of piers and spans which can be most economically built between the two abutments.

An old rule for this case states that the cost of the superstructure must equal the cost of the substructure in order that the cost of the whole may be a minimum. The cost of piers is to be determined by careful surveys and estimates for various locations along the line, while the cost of spans of different length may be approximately ascertained by consulting builders. A comparison of the different possible arrangements determines the most economic plan which sometimes agrees well with this rule.

The cost of common bridges is closely proportional to their weights. If l be the length of span, the formula $W = al + bl^2$ gives a good approximation to the weight (Part I, Art. 45), a and b being constants for the same type of truss. In this, al represents the weight of the track and floor system, while bl^2 represents the weight of the main trusses and lateral bracing. For example, the total weight of iron in pounds in a single-track railroad lattice bridge (not including cross-ties and rails) is about $200l + 7l^2$, if l be the span in feet, while that of a pin-connected bridge is $350l + 5l^2$.

If the cost of piers is about equal, and they be spaced at equal distances apart, the following investigation will give the economic number of spans. Let L be the total distance between end abutments, x the number of spans, and hence $x - 1$ the number of piers, m the cost of the two abutments, n the cost of each pier, and p the cost per pound of the bridge superstructure. The weight of the x spans, each of length $\frac{L}{x}$, is then $x\left(a\frac{L}{x} + b\frac{L^2}{x^2}\right)$, and the total cost of the work is

$$C = m + n(x - 1) + p\left(aL + \frac{bL^2}{x}\right).$$

This will be a minimum when the first derivative of C with respect to x becomes zero, and this gives $n = pb\frac{L^2}{x^2}$, which shows that the cost of one of the intermediate piers should

equal the cost of the main and lateral trusses of one of the spans. Or, $x = \sqrt{\frac{pbL^2}{n}}$ gives the economic number of spans. For example, if $L = 1000$ feet, $a = 350$, and $b = 5$ for pin-connected spans, and $p = 4$ cents per pound, then for $n =$ \$6000, the most economic number of spans is $x = 6$, and the total cost is \$77 300, exclusive of abutments. Here the cost of the piers is \$30 000, and that of the seven spans is \$47 300, which indicates that the old rule may sometimes be at fault. Again, if the cost of a pier be $n =$ \$8000, the economic number of spans is $x = 5$, which gives \$32 000 for the piers, and \$70 000 for the superstructure.

When the cost of piers varies in different parts of the river, the spans will vary in length, the shortest ones generally being nearest the banks. For each possible case a rough estimate of the cost of piers and spans may be made, and thus the arrangement which gives the minimum cost may be determined. For example, suppose the distance between abutments to be 500 feet, a pier near the middle costing \$6000, and piers within 150 feet of the shore costing \$4000 each; then, using the above values of a, b, and p, the cost of one pier and two 250-feet spans would be \$38 000, while the cost of two piers, with a middle span of 200 feet, and two side spans of 150 feet, would be \$32 000.

ART. 10. CHOICE OF KIND OF BRIDGE.

Whether the bridge span is to be deck or through will be determined in each case by the local conditions, among which the grades of the approaches are controlling factors. A deck span is usually cheaper than a through one, since the width of the bridge may be less and something is also saved on abutments and piers, and should hence be chosen if the approaches allow it and proper waterway can be secured beneath it.

The width of the bridge between trusses is determined by the amount of traffic. For a single-track railroad this width for a through bridge is taken as 14 or 15 feet in the clear, while for a deck bridge 10 or 12 feet between centers of trusses is usually enough for short or medium spans.

The cost of the bridge is a material factor in determining the kind which is to be erected, and the problem of selection is hence a very complicated one. For railroads experience has led to the conclusion that at present the best results both as to stability and economy are obtained by using solid rolled beams for short spans up to 15 or 20 feet, plate girders for spans from 15 to 90 feet, riveted lattice trusses for spans from 50 to 150 feet, and pin-connected trusses for spans over 100 feet. It will be observed that these figures overlap each other, indicating that there is no distinct line of demarcation between the length of spans of the different classes, and detailed designs and estimates are often required to determine the cheapest type.

The particular kind of truss is not usually stated in the specifications, this being left to the bidders who often may present plans which differ materially in general appearance. If all these plans conform to the specifications, the contract is awarded to the lowest responsible bidder. The choice of the kind of truss is hence usually made by the sellers rather than by the buyers of bridges, but the question of accepting the tender of the lowest bidder is sometimes influenced by the form of truss adopted in his plan.

The discussion in Art. 4 gives only the general economic conditions which determine the form of truss. The depth of the truss is to be selected so as not only to secure proper headway and afford opportunity for cross-bracing, but also so as to give the least amount of material; this question of economic depth is investigated in Art. 11. The number of panels should be odd rather than even for best economy, and should be such

that the panel lengths, or distances between floor beams, may range from 12 to 24 feet. Probably the best panel length, as far as the floor system is concerned, is that which renders the weight of a floor beam about equal to that of the stringers in one panel.

Æsthetic considerations should not be overlooked in choosing the kind of bridge, and the old maxim that strength, beauty, and economy go together contains some truth. The parabola is a line of beauty, and through trusses having the upper chords broken or curved are among those which now seem to possess the highest degree of economy for spans between 100 and 550 feet. In deck trusses, however, the upper chord is necessarily straight, and the slight downward curvature sometimes given to the lower chord does not appeal to the public as an element of beauty. For deck bridges arches are always more beautiful than trusses, but unfortunately their cost is much greater.

Approximate economic comparisons of trusses of different forms may be made by comparing the theoretic amounts of material, the material in any member being taken as proportional to the product of its maximum stress by its length. Investigations of this kind were first made by WHIPPLE in 1847, and have since proved of value in studying the question of economic proportions. But such investigations are of limited value in comparing the relative economy of different forms, unless the unit stresses for compression be taken less than those for tension, and as required by a formula for columns. To introduce this element in a theoretic comparison leads to great complexity, and it is, in fact, only by making actual designs from a given specification that reliable results can be obtained. The work of BENDER, cited in Art. 7, and CREHORE's Mechanics of Girder (New York, 1886) may be consulted for examples of such investigations.

ART. 11. ECONOMIC DEPTH.

The economic depth of a girder or truss is that which renders its weight a minimum. Such a depth exists by virtue of the facts that the chord material decreases and the web material increases as the depth is increased. For a plate girder it is a rough general rule that the economic depth obtains when the weight of the flanges is equal to the weight of the web. To show this it must be borne in mind that the thickness of the web plate is practically constant for a girder of short span, being rarely greater than $\frac{5}{8}$ nor less than $\frac{3}{8}$ inch. The material in the web hence varies as $a \times h$, and that in the flanges as b/h, where a and b are constants depending on the span loads and working unit stresses. The total material may then be represented by $a \times h + b/h$, which is a minimum when the two terms are equal, that is, when the flange weight equals the web weight, or, more strictly, when the cost of the flanges equals the cost of the web. In practice, however, the weight of the flanges often exceeds that of the web. (See Arts. 66 and 71.)

For a truss an approximate determination of economic depth may be made by computing the stresses in terms of the panel length and depth, multiplying each stress by the length of the corresponding member, and regarding the products as representing the amounts of material, and then finding the depth that renders the sum of these products a minimum. For example, take the PRATT truss of which one-half is shown in Fig. 11. Let the dead load per panel point be w, and the live load $3w$. Let the panel length be p, and the depth of the truss be h. By the methods of Part I the maxi-

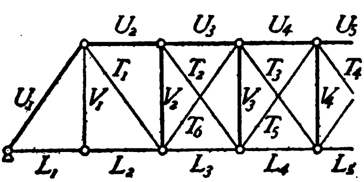


Fig. 11.

mum stress in each member due to the given load is computed, and each stress is then multiplied by the length of the corresponding member. For example,

MEMBER.	STRESS.	STRESS × LENGTH.
L_1	$16\,wp/h$	$16\,wp^2/h$
U_1	$16\,w(h^2+p^2)^{\frac{1}{2}}/h$	$16\,w(h^2+p^2)/h$
V_1	$4\,w$	$4\,wh$
....	....	....

and the sum of all the products in the last column will be found to be

$$\text{Sum} = \left(70\,h + 287\tfrac{2}{3}\,\frac{p^2}{h}\right) w,$$

which represents the amount of material in one-half the truss. Differentiating this expression with respect to h and equating the derivative to zero gives $h = 2.03\,p$, or the theoretic economic depth is about twice the panel length. For this depth the web material, including the end posts, is about 165 wp, and the chord material about 118 wp, the former being about 40 percent greater than the latter. This value of the economic depth is, however, considerably too large for practice, since the investigation has neglected the increase of the amount of material necessary in compression members.

It may be further noted that great exactness in regard to economic depth is not important, since a function changes slowly in the vicinity of a maximum or minimum, so that considerable variations in depth may be made without much increasing the quantity of material. For instance, in the above case the following shows how the material varies for different depths:

Depth h =	1.8	1.9	2.0	2.1 p,
Material =	285.8	284.4	283.8	284.0 wp,

which indicates that the depth may vary ten percent from the economic depth without increasing the material as much as one percent.

Lastly, it may be noted that there has been a constant tendency since about 1875 to build through truss bridges with greater and greater depths. This has resulted from considerations of stiffness as well as those of economic depth. Increasing the depth of a truss diminishes its deflection under live loads and thus decreases the injurious oscillations which wear out a railroad bridge. This tendency is apparent in both long and short spans, but especially in the shorter ones. In some cases the increase in depth has gone so far as to require the vertical posts to be stiffened by horizontal braces placed between them in the plane of the truss and midway between the upper and lower chords.

ART. 12. PRACTICAL CONSIDERATIONS.

The engineer who draws the specifications is primarily responsible both for the strength and security as well as for the economy of the structure. For, if improper working stresses are prescribed, or proper rules for stability are omitted, the builders, under the influence of competition, will present plans of structures lacking in security; or, if excessive and unusual requirements are made in the specifications, the plans presented will not be economical. At present there are so many specifications which may be called standard that it is not possible to go far astray in either of these directions, particularly for railroad bridges. For many highway structures, however, the specifications are very loosely drawn, and every year there are erected some bridges which are defective either in stability or economy. As a general rule economy demands a bridge of proper stability, and the proper degree of stability will be secured by structures of the best economic design.

The designer should, of course, strictly follow the specifications, yet in details and dimensions he has great liberty of choice. He should be well acquainted with the market sizes of materials and with the market prices. Variation from regular sizes always involves delay and extra cost. Uniformity of sizes is advantageous, since several things of one kind can be purchased or made more cheaply than if they are of different dimensions. Simplicity of connections should be studied not only with respect to strength, but also with regard to economy of manufacture. The lines of action of all stresses meeting at a joint should intersect at a point, in order to avoid secondary stresses of twisting or bending. Simplicity, as a rule, leads to both determinate stresses and the economy of material.

In riveted work excessive nicety in the spacing of rivets should be avoided. If possible the pitch should be in even inches, that is either 2, 3, 4, 5, or 6 inches, especially when the rows are long, as in columns and the flanges of plate girders. It will be more economical still if the pitches can be reduced to two, 3 inches and 6 inches, but this is not so easy to attain and still maintain the proper uniform strength throughout.

In pin-connected work it will often be advantageous, particularly for short spans, if the pins are of uniform sizes, except perhaps those at the ends. As the strength of a pin depends more upon its resistance to transverse stresses than to shearing, it is often possible to insure that the prescribed unit stresses shall not be exceeded by properly spacing the eye-bars (Art. 91). Columns and lateral bracing must be arranged with due regard both to economy of shop work and to ease of erection. Field riveting should be reduced to a minimum, since it is more expensive and less satisfactory in regard to strength than shop riveting.

All parts of the metal work of the bridge should be arranged so that they can be easily painted after erection. The shoes,

rollers, and bed plates should be so placed that they cannot become surrounded with dirt from the roadway or approaches. The floor of a highway bridge should be so arranged that water draining from it shall not fall upon the metal work underneath. In short, the designer should endeavor to produce a structure that shall not only be of ample security when erected, but which shall maintain that degree of security through a long life of useful service to the public.

CHAPTER III.

BRIDGE CONTRACTS AND OFFICE WORK.

ART. 13. SPECIFICATIONS.

The local circumstances of the case in hand determine, according to the principles of Chap. II, the number of spans of the bridge to be built, the lengths of the spans, the width of roadway, whether the trusses are to be deck or through, and the character of the traffic. The engineer representing the party that is to own the bridge then prepares rules regarding the loads to be used in the computations, the permissible unit stresses, the quality of the materials, and the character of the workmanship. These rules are called specifications, or sometimes "the specification." All the plans to be submitted by bidders must be in accordance with these specifications, which are afterward made a part of the contract between the buyer and the successful bidder.

Specifications cannot be successfully prepared except by an engineer of experience. In highway bridge work it sometimes happens that county commissioners or town authorities advertise for proposals without having definite specifications, but the result is sure to be that a poor bridge will be erected. Any one can buy a bridge, but only an engineer can do so and obtain both a stable and an economical structure. The highway-bridge specifications of COOPER and those of WADDELL are excellent guides to follow, and they can easily be obtained in pamphlet form. Many railroad companies have their own specifications, and the large bridge companies also have specifications which

they recommend purchasers to follow. The use of such standards by the young engineer will usually result in better work than can be obtained by any specifications prepared by himself.

The following extract from a lecture by THEODORE COOPER states in an excellent manner the fundamental purpose of specifications:

"Their purpose is a twofold one. First: They are to enable bidders upon any work to understand fully the character and extent of the work and what they are expected to furnish and what to do, in order that they may be able to make suitable estimates upon which to formulate an intelligent and proper bid. Second: They are, in connection with the plans, to serve as the reference in regard to all questions as to qualities of the materials and workmanship during the execution of the work, in order to avoid misunderstandings between the engineers and contractors; the contractor not being allowed to furnish poorer or less suitable materials and workmanship than is there specified, nor the engineer to demand any better without giving an extra compensation. Nothing serves better to obtain the best class of contractors and to obviate much of the friction which occurs during construction between the engineer and the contractor than a good specification, carefully and clearly expressed. A loosely drawn and incomplete specification is always attractive to the worst class of contractors, or those who do not intend to do an honest job and who will take advantage of every weak point to get all they can out of the work."

ART. 14. ESTIMATES AND PROPOSALS.

After the preparation of the specifications, proposals or bids are invited from bridge companies for the manufacture and erection of the structure. In general, bridge companies con-

tract for and build only the superstructure, while the piers and abutments are erected by masonry contractors.

The usual mode of procedure is to publish an advertisement which gives the location, length, number of spans, and width of the bridge, stating whether highway or railway, and whether timber, stone, or metal is to be employed. The advertisement mentions where specifications can be seen and information obtained, and names the day and hour when the proposals will be opened. It often states that a certified check for a certain amount must be deposited by each bidder as a guarantee that he will enter into a contract in case the work is awarded to him. Bidders are invited to be present at the opening of the proposals, and the right is reserved to reject any or all bids. It should also be required that each bidder shall present a stress sheet and a general plan of the structure that he purposes to erect.

A bridge company which desires to put in a bid for building the bridge sends one of its agents to the place to procure all the data available. Sometimes the engineer in charge of the work has plans prepared on which the companies estimate and bid, but usually each company prefers to make and submit its own plans. The agent examines closely the locality and estimates the cost of hauling the material from the nearest railroad station, as also the cost of erection. The latter item is often an uncertain one, since delays due to the weather or to floods in streams are liable to arise, and sometimes accidents occur which cause the loss of all profits. It should also be the duty of the agent to become acquainted with the parties who purpose to build the bridge, so that in case of a close competition he may be better prepared to induce them to accept the proposal of the company which he represents.

The computations and designs made by a bidder in order to estimate the cost of a structure are similar to those given in the

preceding and following chapters. The style and proportions of the bridge being decided upon, the stresses are computed by the methods of Part I or Part II, and a stress sheet is prepared, showing these stresses and the sections of the main members. A general drawing is also made showing elevation, plan, and cross-section, with the main features of all details. From this drawing a bill of material is made out, and estimates of the weight and cost of manufacture are prepared. Adding to this the estimated cost of freight and erection, and a fair percentage for interest on invested capital, profit, and contingencies, the bidder decides upon a sum to state in his proposal.

The usual practice in highway-bridge lettings is for each bidder to offer a lump sum for the erection of the superstructure ready for traffic and painted. On railroads it is often the case that the cross-ties, rails, and guard timbers are laid by the railroad company, so that the lump sum is exclusive of the track. On some railroads, however, the proposals are required to be made per pound of the finished structure ready for the track, and in such cases the actual sections of the members are not allowed to exceed by more than 2 or 2½ percent the theoretic sections as required by the stresses and specifications.

ART. 15. LETTINGS AND CONTRACTS.

At the hour stated in the advertisement the proposals are opened and read in the presence of the bidders. The accompanying plans are referred to the engineer in charge to see if they conform to the specifications. It is, however, usually only necessary for him to check the computations of two or three of the lowest bidders if their plans seem otherwise acceptable. On the receipt of the report of the engineer the commissioners or authorities in charge make a formal award of the work to the lowest responsible bidder whose plans are satisfactory, and

he is notified to appear and sign the contract, while the plans and certified checks of the other bidders are returned to them.

It often happens at a bridge letting that the highest bid is about double the lowest. This wide discrepancy is probably due more to the fact that certain companies have better facilities regarding freight and erection than to the relative economy of the several types of trusses. The number of proposals submitted for a structure usually ranges from five to twenty.

This method of bridge lettings, in which each bidder offers his own designs, has many advantages, but it has the disadvantage that only one out of a number of plans is utilized. If twelve bidders each spend $100 in making estimates and designs for a single bridge, there has been expended altogether $1200 which in some way must be paid by the buyers of bridges. It is not an infrequent practice, indeed, that the twelve bidders form a pool, each adding $1200 to his bid, and then the successful bidder pays $100 to each of the eleven unsuccessful ones. This is a necessary evil of the method, perhaps, but the evil is not as great as often assumed, since the expenses of the bridge companies must be paid in some other way if not in this. The expenses of estimating would be lessened if the bidders were limited to plans and designs made by the engineer in charge, but in such cases it usually happens, owing to details of construction, that their bids are higher than for their own designs. Open competition has been one of the elements which has led to the present economic forms of bridge trusses (Chap. II), and, notwithstanding the necessary evils of pools, its results continue in general to be satisfactory.

The contract which is entered into between the parties specifies that the bridge company shall erect the structure according to the plans and specifications, and that the other party shall pay to said company a certain amount for the same. It also sets forth in detail the conditions regarding time of completion

and payment, the liabilities of the contractor for damages due to accidents, penalties for delay of completion, and other conditions mutually agreed upon. When this document is signed both parties are legally bound by its provisions, and the bridge company is ready to begin the detail drawings for the shop work.

A bond is also required to be given by the contractors, signed by them and two responsible bondsmen, binding the contractors under a penalty to execute the contract in pursuance of its terms and conditions, and in accordance with the plans and specifications thereunto annexed. This bond is in law of the nature of a promissory note, and in case of default of the contractors an action at law can be brought to recover the sum stated therein, or such part of it as may be sufficient indemnification for the damages sustained.

There are many engineers who own no bridge works, yet nevertheless bid for and take contracts to erect structures. Such men have arrangements with bridge builders to manufacture their bridges at certain prices per pound, or they make special bargains for the contracts that they secure. Many of these engineers do good work and make a fair profit.

ART. 16. OFFICE PRACTICE.

The engineering department of a bridge company is usually divided into two parts, the estimating or computing division and the detailing or drafting division. The estimating division computes the stresses and makes a stress sheet, giving the principal dimensions and sections, and from this prepares bills of material which enable the amount of its bid to be determined. If it secures the contract, this stress sheet is then turned over to the drafting division, where the details are worked out and the working drawings are prepared. Thus, for the small bridge of

Chap. X, the sheet No. 1 (Fig. 137) is prepared by the estimating division, while the nine other sheets are made by the drafting division. A young graduate on entering the engineering department of a bridge company is generally assigned to the drafting office, where he spends three or four years in obtaining the training that is necessary before he can be promoted to the estimating division.

The working drawings made by the drafting division are for the use of the templet makers, the shop foreman and workmen, and the inspectors. Hence the drawing of each piece should be made so plain and complete that the workmen may clearly and easily understand it. The dimensions of all pieces, rivet spacing, and pitch of rivets should be given in full on the drawings. All printing should be plain and well done, though time should not be wasted in this work. All figures should be large enough to take and show well in the blue print. If the space on the drawings between the rivet heads will not permit of good-sized figures being placed in them, then lines should be projected off to one side of the member and the figures placed between them. Arrow points should be placed at the points between which the distance is given. Quite heavy lines should be used so as to give a good clear blue print, while fine ones should be avoided except for dimension lines.

The data which the draftsman receives from the computing division consist of the stress sheet showing the stresses in the members and the sections to be used, and a copy of the specifications. The first thing the draftsman generally does is to find out what material is required and how much of it. If the structure is of considerable size, this is best done by laying out the work in a general way on thick brown paper prepared for this purpose, not stopping to put in the details, but going far enough to enable him to determine quite closely what are the lengths and sizes of the angles and plates which are required. He then

consults the list of material in stock, and if he finds any in his bill that is not in stock he makes out an order list, from which the material is ordered immediately by the purchasing department, for it must be on hand as soon as the drawings are finished.

The drawings already laid out in a general way are now completed by placing tracing linen over them and tracing the work from the paper and filling out the details on the tracing linen. In making the details many computations of rivet connections must be made, so that the work shall compare to both the specifications and the practice of the bridge company. The detailer must have a good knowledge of the mechanics of materials in order to be successful in his work.

The tracing is done on the back or unglazed side of the linen. This side shows pencil lines much better than the glazed side, and it will take ink lines just as well. When it is necessary to do any erasing on the tracing linen a rubber ink eraser is used carefully and patiently. The erased area is then rubbed with a stick of pumice stone before inking again, to prevent the ink from spreading. The point of a knife or other sharp tool should not be used to erase lines or spots from tracing linen which have to be inked over again. If the surface of the tracing linen becomes greasy so that the ink will not take well, a little powdered chalk, sprinkled on and rubbed carefully with a cloth, absorbs the grease and gives a better working surface.

After the drawings have been completed a bill of material is prepared. This is made in such a way as to serve as a shipping list also. It contains a list of every individual piece entering into the structure. These are arranged in groups in the list just as the pieces are assembled to make up a member. All pieces which require forging, such as eye-bars, ties, and

counters, are listed also on a sheet known as the forge sheet. Full dimensions and details, and perhaps sketches, are required on this sheet, which then goes to the forge shop. A list of all field rivets and bolts is also made, which gives their size, length, grip, and their location in the bridge.

After the listing has been done both drawings and lists go back to the computing room, where every item, line, and figure is carefully checked. If no errors are found, which is rarely the case, the draftsman may consider his work completed, but if any are found they must be corrected. Blue prints are next taken from the tracings, and the work is ready for the shops.

In the drafting rooms of some of our larger bridge plants there are as many as twenty-five or thirty men, all under the immediate charge of a superintendent or head draftsman, who is thoroughly posted on all kinds of detail work and shop methods. Each man is supposed to be supplied with a complete outfit of drawing tools, and to have a desk to himself, with drawers for paper, tracing linen, and tools. The old style of drawing desk is flat on top and from three feet six inches to four feet in height, with a regular drawing board on top. The more modern desks are not quite so high, and the top is so arranged that it can be tipped up toward the draftsman, making it easier to see and get at all parts of the drawing.

Adjoining the drafting room is a fireproof vault in which are kept all plans and drawings of structures that have been built by the company. These are of great value to the company and also to every draftsman. The vault is the draftsman's library. In consulting it he may find many unique and useful designs and details which will greatly facilitate his work, especially in unusual connections such as occur in skew bridges. A young draftsman should also take advantage of every opportunity to observe and study shop processes in order to be able to see the reasons of the rules of the company regarding bridge

details. The young engineer who does not understand the reasons that govern his work cannot make good and satisfactory drawings for his employer, but he who knows the theory and practice of the subject will do the best work and earn the most rapid promotion.

ART 17. RULES FOR SHOP DRAWINGS.

The following rules for making shop drawings are those given by the American Bridge Company in its Standards for Structural Details, 1901. They are here printed by permission kindly granted by C. C. SCHNEIDER, Vice President. These are general rules applicable to all kinds of detailed drawings; other special rules for drawings of plate-girder bridges, truss bridges, and buildings are also given in the volume above mentioned.

The standard size of sheet shall be 24 by 36 inches, with two border lines $\frac{1}{2}$ and 1 inch from the edge respectively. Small sheets shall be used for beams, pins, eye-bars, etc. Special forms are provided for these sheets.

The title shall be arranged uniformly for each contract near the lower right-hand corner of the sheet. A stamp is provided for the contract, sheet number, etc. It shall be applied in the lower right-hand corner of the sheet. The name of the draftsman in charge of the work shall appear in full, others with initials only.

Detail drawings shall as a rule be made in scale $\frac{3}{4}$ or 1 inch to the foot; for large plate and lattice girders $\frac{1}{2}$ and $\frac{5}{8}$ inch may be used. Larger scales, such as $1\frac{1}{2}$ and 3 inches to the foot, are permissible only for showing certain complicated details or for machine work. Large sheets shall be neatly and carefully made to exact scale.

Members shall be detailed in the position which they occupy in the structure, that is, horizontal members shall be shown

lengthwise, and vertical members crosswise, on the sheet. Inclined members (and vertical ones when necessary on account of space) may be shown lengthwise on the sheet, but then always with their lower end to the left. Avoid notes as much as possible. Where there is the least chance for ambiguity make another view.

Show all elevations, sections, and views in their proper position — looking toward the member. Place the top view directly above and bottom view below the elevation. The bottom view shall always consist of a horizontal section seen from above.

In sectional views the web or gusset plates shall always be blackened. Angles, fillers, etc., shall be cross-hatched, but only when necessary on account of clearness. In a plate girder, for instance, it is not necessary to cross-hatch all the stiffeners and fillers in the bottom view.

Holes for field connections shall always be blackened, and shall, as a rule, be shown in all elevations and sectional views. Rivet heads shall be shown only when necessary; for instance, at the ends of members, around field connections, when countersunk, flattened, etc.

In detailing members which adjoin or connect to others in the structure, part of the latter shall be shown in red, sufficiently to indicate the clearance required or the nature of the connection. Plain building work is exempt from this rule.

When part of one member is detailed same as another, figures for rivet spacing, etc., shall not be repeated; refer to previous sheet or sheets, bearing in mind that these must contain final information. It is not permissible to refer to a sheet, which in turn refers to another. Main dimensions, which are necessary for checking, such as center-to-center distances, story heights, etc., shall be repeated from sheet to sheet.

Holes for field connections must always be located independently, even if figured in connection with shop-rivets; they shall be repeated from sheet to sheet unless they are standard, in which case they shall be identified by a mark and the sheet given on which they are detailed.

A diagram in small scale, showing the relative position of the member in the structure, shall appear on every sheet. The member or members, which are detailed on the sheet, shall be shown in black, and the rest in red, ink. Plain building work is exempt from this rule.

The quality of material, workmanship, size of rivets, etc., shall be specified on every sheet as far as it refers to the sheet itself. Standard workmanship, such as milling and tight fit of stiffeners, milling ends of columns, etc., shall not be specified on drawings.

Each piece which is shipped separately shall have a shipping mark. These marks shall consist of capital letters and numerals, or numerals only; no small letters shall be used except when sub-marking becomes absolutely necessary. The letters R. and L. shall be used only to designate "right" and "left." Never use the word "marked" in abbreviated form in front of the letters, for instance, instead of "3 Floorbeams, mk. G4," say "3 Floorbeams G4."

Pieces which are shipped bolted on to a member shall, as a rule, also have a separate mark in order to identify them should they for some reason or another become detached from the main member. The drawing shall specify which pieces are to be bolted on for shipment, and the necessary bolts shall be billed.

A system of assembling marks shall be established for all small pieces in a structure which repeat themselves in great numbers. These marks shall consist of small letters and

numerals, or numerals only; no capital letters shall be used; avoid prime and sub-marks, such as m'a.

For all lettering use plain letters. For title, main dimensions, and for all marks, particularly shipping marks, use heavy type. Red ink (Winsor & Newton's carmine) shall be used for dimension, reference lines, etc.

Conventional signs for rivets are shown on page 18 [Standards for Structural Details]. Countersunk rivet heads project $\frac{1}{8}''$; if less height of heads is required, drawings shall specify that they are to be chipped or that they must not project more than $\frac{1}{16}''$. Flattened heads project from $\frac{3}{8}''$ to $\frac{7}{16}''$; if less height of heads is required, they shall be countersunk.

Metals in section shall be shown as follows:

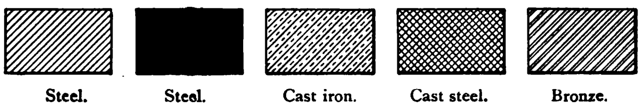


Fig. 12.

Shop bills shall be written on special forms provided for the purpose. When the bills appear on the drawings as well, they shall either be placed close to the member to which they belong or on the right-hand side of the sheet.

When the drawings do not contain any shop bills, these shall be so written that each sheet can have its bills attached to it, if desired; that is, one page of shop bills shall not contain bills for two sheets of drawings.

In large structures, such as elevated railroads, viaducts, etc., which always are subdivided into shipments of suitable size, both mill and shop bills must be written separately for each shipment.

In writing the shop bill, bear in mind that it shall serve as a guide for the laying out and assembling of the member, besides being a list of the material required. For this reason members

which are radically different as to material shall not be bunched in the same shop bill, neither shall pieces which have different marks be bunched in the same item, even if the material is the same.

The main material in a member shall be billed first, followed by the smaller pieces. It is generally a good practice to begin at the left end of a girder, or at the bottom of a post or column. Do not bill first all the angles and then all the flats; when, for instance, the end stiffeners in a girder are billed, the fillers belonging to them shall follow immediately after the angles, and so on. In a column each different bracket shall be billed complete by itself.

When machine-finished surfaces are required, the drawing and the shop bill shall specify the finished width and length of the piece, proper allowance for shearing and planing being made in mill bill. When the metal is to be planed as to thickness, the drawing and shop bill shall specify both the ordered and the finished thickness, for instance, one pl. $12'' \times \frac{13}{16}'' \times 1'6''$ planed to $\frac{3}{4}''$.

Flats and universal plates over $4''$ in width should be ordered in even inches; flats under $4''$ should be ordered by $\frac{1}{2}''$ variation in width. Flats $\frac{1}{4}''$ and under in thickness are very difficult to secure from the mills, and should be avoided if possible.

Every contract embracing different classes of work shall have a subdivision for each class. These subdivisions will be furnished by the chief engineer of the district. Drawings, shop, and shipping bills must be kept separate for each division.

CHAPTER IV.

BRIDGE SHOPS AND SHOP PRACTICE.*

ART. 18. GENERAL CONSIDERATIONS.

The plant of a bridge company consists of its offices and shop buildings, the tools and machinery necessary for converting the rough material into the finished product, together with the appliances for handling and treating the material at all stages of the process. There is also the erecting apparatus, which is used for putting up the bridge or other structure in the place for which it was designed.

Each building or shop is under the immediate charge of a foreman or superintendent, the entire plant being in the care of the general superintendent or manager. He has the general supervision of all work, and each day receives from his foremen reports of what has been accomplished. He must keep in touch with every contract the company has in hand, and at all times know exactly how it stands. He should know every man in the plant and be familiar with his record. He must understand every wrinkle of shop practice, and be a good business man. On the general superintendent depends, in the greatest degree, the earning power of the plant.

All work in the shops is done on a program made out by the general superintendent, which prescribes the approximate dates when the various pieces of work in hand are to be completed. Every effort is directed toward adhering as closely as

* By THADDEUS MERRIMAN, C.E., Bridge Inspector in the United States for the Guayaquil and Quito Railway.

possible to this program, to the end that all of the various shops shall be kept continually busy, and that the parts of the bridges being built shall be completed in the order in which they are required in the field.

The shops should be so arranged with respect to each other that the motion of the material in passing through them will be continuous and in one direction. The rough material should come in at one end of the plant and the finished product pass out at the other. In a plant so arranged the time lost in handling the material will be a minimum, as also, other things being equal, the cost of production.

The buildings should be connected by narrow-gage tracks running lengthwise through them, so that material when loaded on trucks can be taken from one part of the plant to any other quickly and with no additional handling. Between the buildings, and around them, as well as at each end of the plant, ample yard-room should be provided. Here will be stored the rough material, the partially completed material awaiting its turn to go to the assembling shop, and the finished product waiting to be loaded for shipment. These yards should be carefully laid out, so that the carrying distance to the nearest track will always be a short one, and whenever possible it should be made by an overhead traveling crane. Fig. 13 shows such a crane made by Pawling & Harnischfeger, Milwaukee, Wis. This crane consists essentially of two plate-girders side by side, and mounted on wheels which run on elevated tracks. On the top flanges of these girders there is a trolley, which can move from one end of the girders to the other. This trolley carries the hoisting motors and drums by means of which the loads are lifted when attached to the hooks.

All motions are controlled by a man in the cab on the right of the picture. The entire crane can move backward and for-

ward and the trolley from one side to the other, thus reaching every point between the main tracks. The crane shown is in the yard of the Pennsylvania Steel Company at Steelton, Pa., and its capacity is twenty tons. It is also provided with an auxiliary hoist, by means of which small loads can be lifted more rapidly than if picked up by the large hooks, as the latter moves at a constant speed for all loads.

Some bridge companies own their furnaces and roll their own material; while others, on the receipt of a contract for a structure, buy the material necessary for manufacturing it. In either case the rough material is stored in the yards until the shops are ready to begin work on it.

The railroad facilities, both for bringing material to the plant and for shipping away the finished product, constitute an important factor which should be carefully considered in studies for the location of a new plant. That company which has the choice of the greatest number of routes of shipment can obtain the lowest freight rates.

The various buildings or shops should be large, light, well heated and ventilated, and of fire-proof construction. Their roof trusses should be made sufficiently heavy to carry the ordinary roof loads, together with numerous small overhead travelers, which can reach to every part of the floor space. The large overhead traveling cranes which handle the material in the assembling shop after it has begun to assume form and weight, are entirely independent of the building, each one having its own supports.

Of the shops and buildings which go to make up the plant of a bridge company the following are the most important: The Power Plant, the Pattern and Templet Shop, the Shear and Punch Shop, the Assembling Shop, the Machine Shop, and the Forge Shop.

Fig. 13. Twenty-ton Traveling Crane in Yard of Pennsylvania Steel Company.

ART. 19. THE POWER PLANT.

The present tendency in modern manufacturing establishments is toward a centralization of the engines and apparatus which generate the power required in the different buildings. Large batteries of boilers may be installed which will furnish steam through short lengths of pipe to engines of great horse-power and high efficiency. These engines drive electric dynamos, which, in turn, through electric motors located at convenient points about the plant, keep in motion lines of shafting, or the motors themselves may be directly connected to the machines to be driven.

The advantages of this system may be summed up briefly, as follows: A cheap grade of fuel may be used under the boilers, and mechanical stokers will take the place of and greatly reduce the number of firemen necessary for a scattered system. The number of engine attendants is reduced, their places being taken by the electricians required to keep the wire lines and motors in shape. Power can be conveyed to every part of the plant with a minimum of loss, as it is used only in performing useful work. The first cost of such a system may be very high, but the rapidity with which it is displacing the older methods is abundant proof that it is the most economical.

The power plant of a bridge company contains, then, the boilers for generating steam, the engines which drive the dynamos that furnish power and light to all parts of the works, and the engines driving the air compressors that deliver the air under pressure through pipes to many convenient points throughout the plant. From these points the air is carried through armored rubber hose and furnishes power to the portable pneumatic drills, riveters, hammers, and reamers. It is also used for furnishing draft to the rivet furnaces and blacksmith forges, for cleaning the surfaces of the finished material

preparatory to painting, and finally for applying the paint to them by means of a spray.

The hydraulic pumps and accumulators which supply the water to the hydraulic riveters and presses would also be located in this building, if the idea of centralization were carried to its limit.

The machinery in the power plant should be in duplicate, so that an accident may not result in shutting down the entire plant. The electric motors about the plant should as nearly as possible be of the same capacity and type, so that their various parts may be interchangeable.

ART. 20. THE PATTERN AND TEMPLET SHOP.

A pattern is a full-sized model of anything which is to be made either of cast iron or of cast steel. It is usually made somewhat larger in size than the finished piece is to be, so as to allow for shrinkage and for machining the rough surfaces. A pattern is generally made of wood and in parts, so as to facilitate the making of the molds into which the metal is to be poured.

A templet is either a board or a framework of boards, the plane of one of its sides being an exact representation of the plane of one of the sides of the metal shape or piece which is to be made. This templet is clamped upon the piece, and then the positions of the holes, bevels, and notches are marked upon the metal face. The centers of the holes are marked by striking with a hammer a center punch which snugly fits the holes in the templet, the position of each hole being thus indicated by a small indentation in the metal. The bevel and notch lines are drawn on the metal by scratching or marking with chalk along the edges of the templet.

The machinery in this shop consists of various wood-working machines, such as buzz-saws for ripping and cross-cutting, planers and borers, both single and multiple, together with all tools usually found in a large carpenter shop. This shop should be isolated from the other buildings of the plant, as the inflammable nature of its contents makes the danger of fire a constant menace.

The templets for much of the work, especially that requiring great nicety of fit, are laid out in full size on the floor of the shop, while for small and general work each templet is made separately from the detail working drawings furnished by the engineering office, and which give all clearances and dimensions. This latter method has the disadvantage, however, of giving no check on the work of the drafting room, but errors there should be of infrequent occurrence.

As many parts as possible of each structure should be duplicates, so that the number of templets required will be a minimum; the rivet spacing should always be made as uniform as possible, and all other details in the design made as simple as practicable, so that both the making of the templets and the putting together of the work will be facilitated and simplified.

Templets are usually made of soft white pine about $\frac{7}{8}$ inch thick. This material should be thoroughly seasoned, so that there will be no shrinkage before the templets are laid out on the metal.

As the various members and parts of a bridge are made in different parts of the plant, it is necessary that all the shops should work to the same standard of measure. Many companies have their own standard, which has been made by them and is considered to be absolute for all of their work. It is usually made of selected and well-seasoned white pine, and thoroughly coated with varnish so as to prevent any changes

in length due to moisture. It is from twenty to twenty-five feet long, mounted permanently on legs and graduated on its upper edge at some known temperature, usually about 70° F. All tapes, rods, and poles used about the plant are from time to time compared with this standard and corrected when necessary.

Since 1890, however, the steel tape, of which there are many kinds and varieties, has been coming into general use. These tapes are usually graduated to inches and sixteenths, and are guaranteed by their makers to conform to the United States standard when at a known temperature and under a given tension. The principal advantages derived from the use of the steel tape are, that, as it is made of the same material as that which is being measured, the temperature errors are small, and also that in measuring a long piece its total length can be determined at one reading. But a steel tape will stretch under constant use, and from winding and unwinding on its reel. If such a tape be adopted as a standard, it should be carefully put away and never used except for standardizing the similar tapes used in the shops.

The American Bridge Company, which operates a large number of plants, has adopted as a standard the tape manufactured by George M. Eddy & Company, who give the constants of this tape as follows: cross-section, $\frac{1}{2}$ inch wide and 0.008 inch thick, coefficient of expansion 0.0000067 for one degree Fahrenheit. This tape is guaranteed by the manufacturers to conform to the standard of the United States government when at a temperature of 62° F., and under a pull of twelve pounds.

ART. 21. THE SHEAR AND PUNCH SHOP.

In this shop the various plates and shapes which go to make up the members of the bridge are straightened, cut to length and bevel, and have the necessary holes punched in them.

The plates and shapes as they come from the mill are often slightly buckled or out of line, and before any work is done upon them they must be made perfectly plane and straight, as otherwise great difficulty will be had in assembling them. Plates are straightened by passing them through a series of rolls, usually consisting of six. These rolls are so arranged that the vertical distance between them can be nicely adjusted, and also so that this distance can be made less at one end than at the other. By this means one side of the plate can be stretched, and imperfect alignment of its edges corrected, by repeatedly passing it under the rolls, which can be turned in either direction. Bends or buckles in the plates are removed by the vertical pressure exerted by the rolls. Fig. 14 shows such a plate-straightening machine, as made by the Hilles & Jones Company, Wilmington, Del.

Beams, channels, zees, and other odd shapes, are best straightened by applying local pressure to the parts out of line, either by a screw press or some form of power-driven cam acting on a plunger. Angles, however, are more readily straightened by passing them through a series of grooved rolls, which act similarly to those described above for straightening plates. When a piece is so badly out of line that these methods will fail to straighten it, — and this is particularly true of plates, — it must be heated and hammered, or else hammered cold; but this latter should seldom be done, as it is very destructive to the material.

When a piece is to be curved, this is now done. Curves of long radius are made by bending the material under the presses, the holes having previously been punched in it. When, however, the radius is short, the metal is heated, and the bend made by hammering while hot, and the holes then punched so that they will be in proper position. If it be necessary that the curve in the finished member shall be an exact one, the material is first curved, and the holes then drilled, so as to avoid the dis-

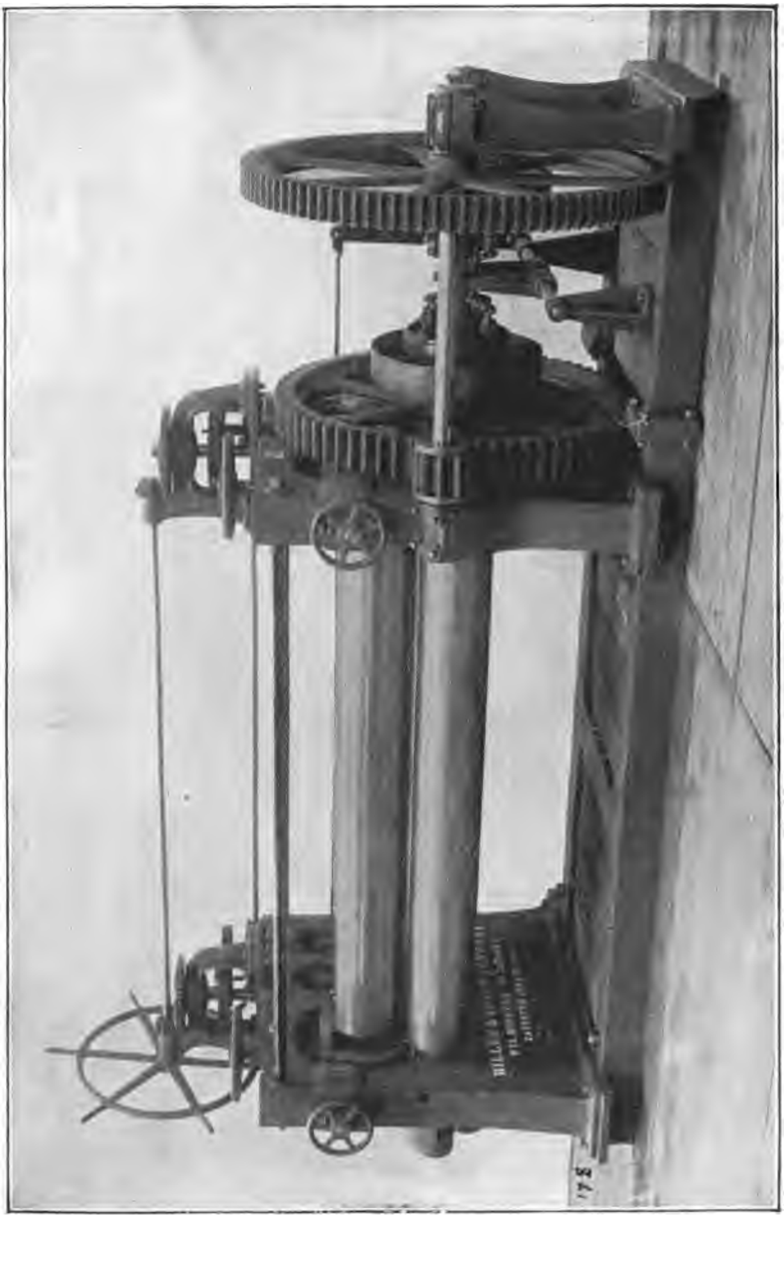


Fig. 14. Plate Straightening Machine.

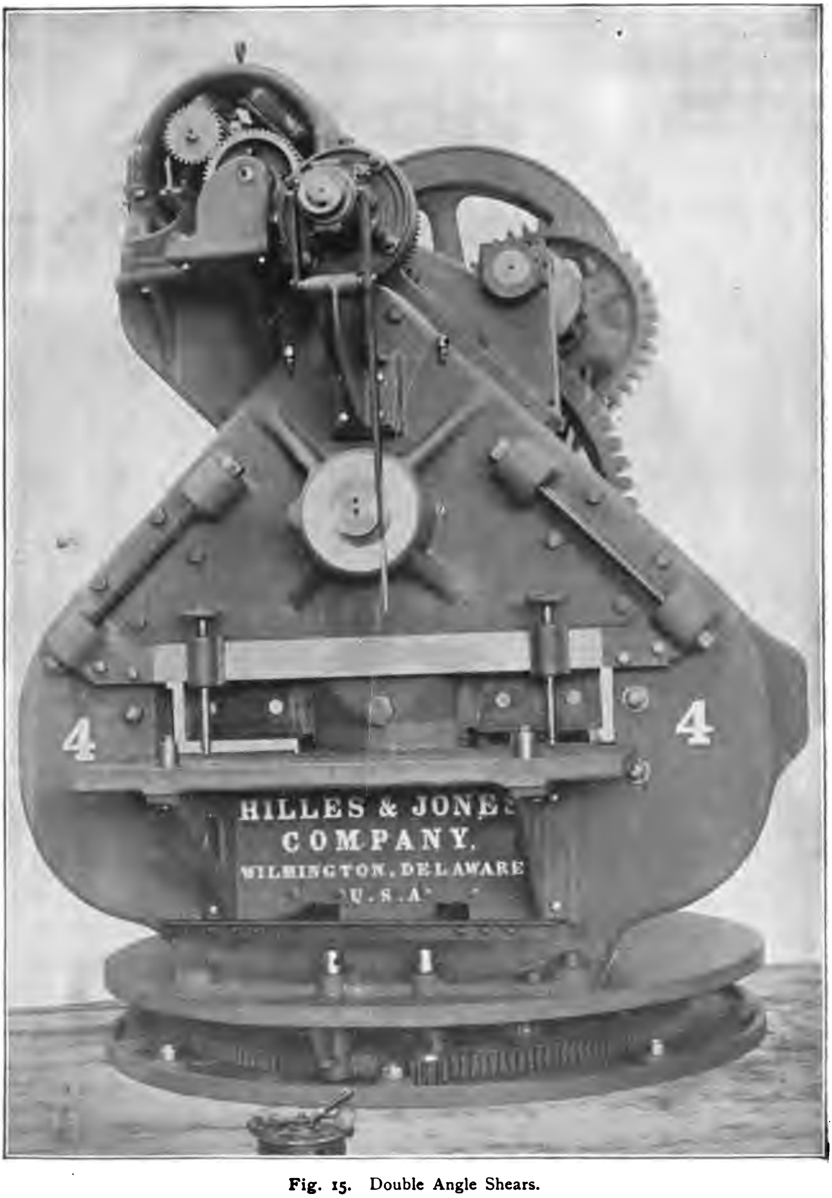


Fig. 15. Double Angle Shears.

tortion due to the punching. But where the very best work is required, all the parts of the member are assembled, clamped together in their proper positions, and the holes then drilled in place. By this method no chance is given the different parts to spring away from their exact positions.

Most of the material usually comes from the mills in multiple lengths, and the first operation is to cut it to the lengths required. The templets are clamped into place, and the positions of all holes, bevels, and cuts, marked off. Many of the pieces are cut to exact length at once; but those whose ends are later to be machined are usually cut $\frac{1}{4}$ inch longer.

All plates and angles are cut by means of a shear. A movable knife is forced down with great pressure upon the material to be cut, which rests upon a stationary ledge, the result being that the metal is cut or broken between the planes or edges of the knife and the ledge. In practice, the edge of the knife does not lie exactly in the plane of the ledge, but is placed as nearly as possible in that position. The more nearly these planes coincide, the squarer and cleaner will be the resulting cut.

In shearing angles the knife begins cutting in the fillet of the angle, and then shears each leg equally. Skew cuts can be made as readily as square ones, but can only lie in the vertical plane in which the shearing knife moves. No shear can make a reëntrant cut without distorting the material and ruining it.

Fig. 15 shows a double angle shears made by Hilles & Jones Company. This is a motor-driven machine and can make both right and left cuts, thus obviating the necessity of turning the piece end for end. Angles placed in the knives on the left-hand and right-hand sides of the machine respectively will be cut like the pieces marked 'left' and 'right' in Fig. 16.

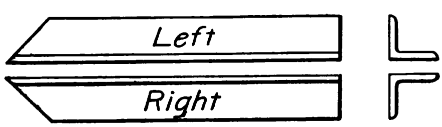


Fig. 16.

The whole machine is mounted on a turn-table, thus doing away with the necessity of slewing the angles across the shop whenever a skew cut is to be made. The machine as shown is not set up ready for use. When installed in the shop, all of the turn-table arrangements are hidden by the floor.

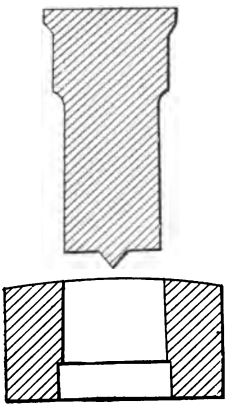


Fig. 17.

The material having now been cut to length and otherwise prepared, it is ready to have the holes, through which the rivets are to be passed, punched in it. These holes are made by forcing a circular rod or "punch" of hard steel through the metal. Fig. 17 shows such a punch with its die.

The tit on the bottom of the punch is for the purpose of centering it over the indentation previously made by help of the templet, and which indicates the center of the hole to be made. Most specifications require that the diameter of the die shall not exceed the diameter of the punch by more than $\frac{1}{16}$ inch. This is required in order to insure good smooth holes without ragged edges. Almost any shape of hole can be made by a punch, but in bridge work the need for odd-shaped holes is very slight.

There are many varieties of punching machines, each designed for doing a different class of work. Single punches for both large and small work, plate punches which can reach to the centers of the largest plates, and multiple punches which can punch several holes at one time are the general types usually found in a bridge plant. Fig. 18 shows a single punch made by Hilles & Jones Company. This is a motor-driven machine which can take a plate 36 inches wide and has a maximum capacity of punching a $1\frac{1}{4}$ inch hole through 1 inch of medium steel.

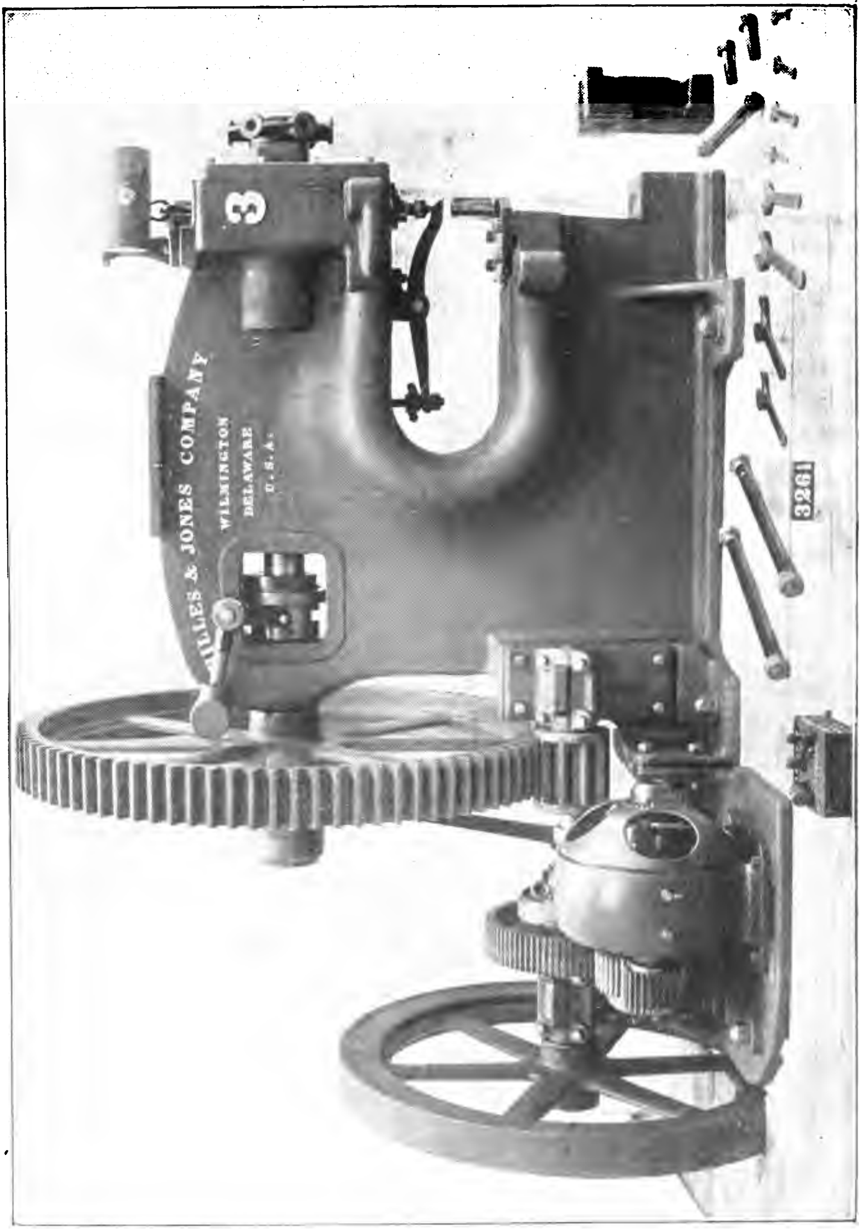


Fig. 18. Single Punch.

Multiple punches are especially useful for work on the webs of floor beams and stringers, and on any plates which require a large number of holes regularly and systematically placed. A number of angles or other shapes which are to be similarly punched can also be passed through one of these machines at the same time. Briefly, their operation is as follows: the plate or shapes to be punched are attached to a carriage which is moved by means of a rack and pinion and which can be stopped at regular intervals, usually multiples of $1\frac{1}{2}$ and 2 inches. If this carriage be moved forward, the plates or shapes will be brought under the punches and the longitudinal spacing of the holes will be determined by the successive positions of the carriage. The transverse positions of the holes are fixed, either by moving the punches on their frame or by attaching the material to different points on the carriage. Such punches can be made of any size and to punch any number of holes; the type generally used, however, will take a plate six feet wide and punch 18 holes in this width at one stroke.

Multiple punches do very good work, and there is no tendency to cumulative error, as the spacing plates on which the carriage depends for its successive positions are very carefully made and cannot become deranged.

When the thickness of the iron or steel to be punched exceeds the diameter of the punch, the results obtained are not good. The material punched is unduly stressed, and the holes are not straight and smooth. Thick plates and shapes must therefore be drilled, but this is a slower and more expensive operation. Single and gang drills are used for this work, but the percentage of drilled to punched holes in ordinary bridge work is very small.

The hole made by a punch is not cylindrical, but is tapering, the diameters at the edges being the diameters of the punch

and the die respectively. Specifications usually require that all punched holes, and especially those in tension members, shall be reamed to a diameter $\frac{1}{8}$ inch larger than the punch, so that all material injured in the process of punching will be removed.

Wrought iron, soft steel, and medium steel can be easily punched, but hard steel must always be drilled, as it cannot be punched without cracking.

It has been shown that a shear cannot make a reëntrant cut, and, as many shapes have several such angles, some other method of cutting them is necessary. A shear can be designed for cutting every shape commonly rolled, but the great variety of machines which would be required, and the consequent greater division of the work, forbids their use. All shapes except angles are therefore cut by the cold saw, which consists of a round disc of hard steel about $\frac{1}{4}$ inch thick and about 40 inches in diameter. Small notches are cut into this disc so as to roughen its periphery, and it is driven at about 2000 revolutions per minute. The shape to be cut is firmly clamped into place, and the saw is slowly fed forward and into it. The heat generated by the contact of this rapidly revolving disc sufficiently melts the material of the shape so that the saw passes on through it. Square and skew cuts can be made with equal facility, being limited only by the diameter of the saw and its forward feed.

As has already been indicated, the machinery in this shop consists of the straightening rolls and presses, the shears for both plates and angles, a variety of punches and drills capable of doing all classes of work, and the cold saw for cutting and beveling shapes.

Overhead cranes and hand travelers are provided for easily and rapidly moving the material at all times to and from any part of the shop. There also are found the plate-planing

machines for planing the sheared edges of plates, to remove the material injured in shearing, and a number of small planers which are used for such work as rounding the corners of angles so as to make them fit into the fillet of another.

The material, having passed through this shop, is in every way ready to be brought and fitted together preparatory to being made into the final members by means of rivets. It may at once be sent to the assembling shop, or else await its turn in the yards. Those pieces which require no more work and are to be shipped loose are now sent to the shipping yard, where they are made ready to be forwarded into the field.

In this shop, each piece, as the work on it is completed, is marked with a letter or number, which designates its position in the finished member, and all pieces which belong to the same structure are marked with the job number, so that they may be readily kept separate and identified in the assembling shop.

ART. 22. THE ASSEMBLING SHOP.

In this shop all the material which goes to make up the various members of the structure is brought together, fitted, riveted, and made of exact dimensions; when the members leave this shop, they are ready to be prepared for shipment.

In general, this shop is laid out so that all of the heavy work is done on one side, the small work being done on the other. The great diversity of the work done by a bridge company, however, renders such a strict division sometimes impossible. On one side of the shop is found the solid riveted work, such as plate girders, floor beams, and stringers, while on the other are the posts and chord sections of truss bridges, steel building work, and lateral members. Fig. 19 is a view of the assembling shop of the American Bridge Company at East Berlin, Conn., taken with the camera standing at the finishing end of the building.

The various parts of each member which have already been made ready for assembling are now brought together into their proper positions and securely fastened with bolts. COOPER's Specifications for Steel Railroad Bridges require that all punched rivet holes shall come together so that "a rivet $\frac{1}{16}''$ less in diameter than the hole can generally be entered hot into any hole without reaming or straining the metal by drifts." In general, however, all holes are punched small, and, after assembling, are enlarged by reaming, thus insuring good

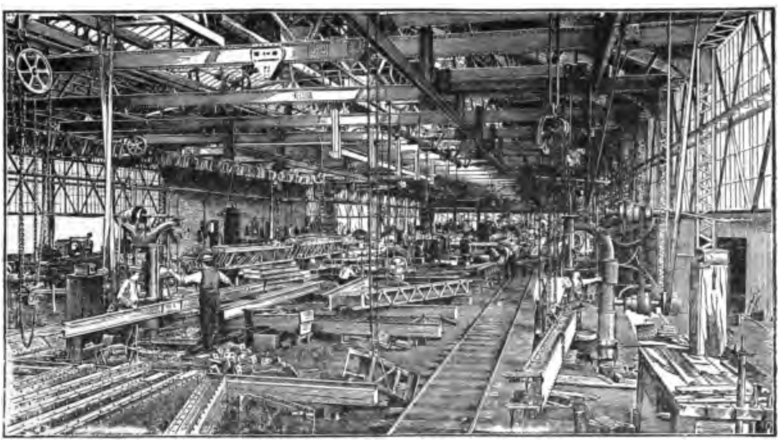


Fig. 19.

smooth holes besides removing all of the material injured in the process of punching. Before assembling, those surfaces of the metal that are to be in contact should be thoroughly painted, and as this is the only paint which can ever be put on them, it should be very carefully applied.

The drift pin, which is a tapering pin of hard steel, is now used for bringing the various parts into their proper positions preparatory to bolting them together. When the parts are not rigidly held, no bad effects can result from its use, but when much riveting has been done on a member and the parts are

securely held, the drifting can only result in causing severe local stresses, and may even rupture the material.

Two general types of machine are used for reaming or enlarging the punched holes after assembling and preparatory to riveting. These two machines are shown in Figs. 20 and 21. The first, made by The Pond Machine Tool Company of Plainfield, N. J., is known as a radial drill. Any number of these machines may be set up side by side; Fig. 20 shows four so arranged, all being driven from the same shaft.

The drills or reaming tools, as the case may be, are mounted on a pivoted arm, and can themselves be moved back and forth on these arms so that they can reach to any part of the half circle described by the pivoted arm. Large pieces of work are brought to these machines and all holes within reach of the arms are reamed or drilled. In this way many drills can be kept moving at the same time without interference, and they can also be rapidly moved from one position to another. The necessity of moving the piece on which the work is being done is also avoided.

The portable rotary reamer and drill shown in Fig. 21 is made by the Philadelphia Pneumatic Tool Company. It is in reality a compressed air turbine, the drill or reaming tool being attached directly to its shaft. Two men are required to operate it, and it can be used in almost any position. Power is furnished by compressed air, which is delivered to the machine through armored rubber hose.

All holes having now been reamed, the member is carefully examined to see that everything is right and in its proper position before it is passed on to be riveted up.

There are two general types of riveting machines, the hydraulic riveter and that driven by compressed air. Riveters operated by hydraulic power are usually stationary. Fig. 22

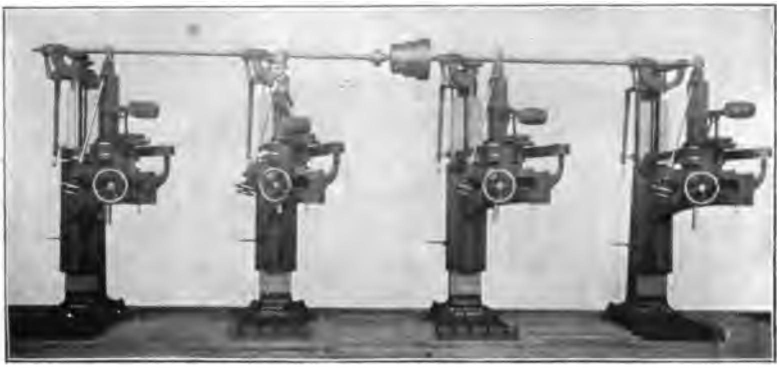


Fig 20. Pond's Radial Drill.

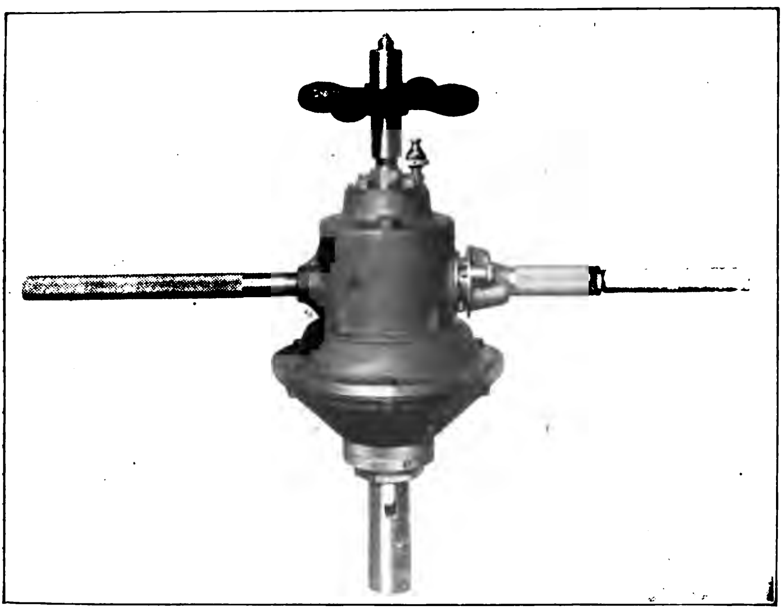


Fig. 21. Pneumatic Rotary Drill.

shows such a machine made by William Sellers & Co., Philadelphia, Pa. As seen in the shop, all of that portion of the machine below the floor line is invisible. The depth of the jaws is usually about ten feet, so that the largest plate girders can be taken; in fact, these machines are used almost exclusively for girder work. The piece to be riveted is carried by an overhead crane, which is controlled from a point near the riveting machine. All motions are provided for in this crane, so that the piece can be readily and easily brought into any position between the jaws of the machine. The hydraulic cylinder on the left operates the riveting bar, and is controlled by the lever shown on the extreme left of the machine.

When the member to be riveted exceeds ten tons in weight, it is usually more economical to move portable riveters about it than to bring it to the stationary machine, but this limit depends not so much on the weight as upon the capacity of the cranes which must handle it.

Hydraulic riveters do the best work; they are simple, do not easily get out of order, and always exert their full pressure upon the rivet being driven. Various forms of portable hydraulic riveters have been made, but they are cumbersome affairs and have not proven very successful.

The riveting machines driven by compressed air are of two distinct forms. In the first, the piston of the air cylinder is attached to a toggle joint, which operates the riveting cup. Except for small work this style is not now generally used. Unless very carefully adjusted, it will never work up to its power; and, as no two consecutive pieces of work to be riveted are of the same thickness, this adjustment becomes a very important factor in securing uniform and tightly driven rivets. In the other form of air riveter the air piston is attached to a plunger, which compresses oil in another cylinder; this oil

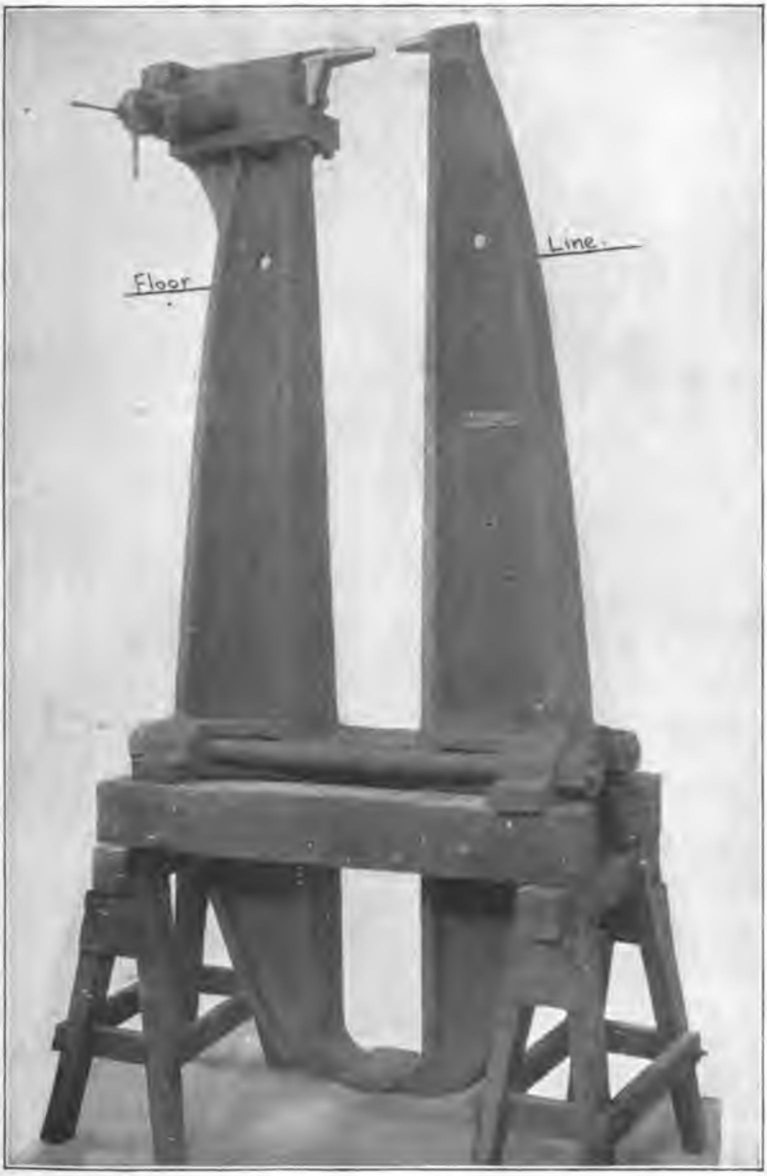


Fig. 22. Hydraulic Riveter.

in turn driving forward the piston which carries the riveting cup. Riveters of this type do excellent work, and always, unless there be continued leakage of oil, produce their full pressure when the air valve is opened wide. No adjustment other than the initial quantity of oil is needed, and even with slight care this will remain constant. Such a machine made by Pedrick and Ayer, Philadelphia, Pa., is shown in Fig. 23.

In every structure there are some rivets which cannot be reached by the power machines. These must be driven by hand, and this is a slow and expensive operation. The comparatively recent introduction of the compressed-air hand-hammer has, however, greatly reduced the cost of this work. Such a hammer, made by the Philadelphia Pneumatic Tool Company, is shown in Fig. 24. These hammers strike a large number of rather light but very rapid blows, and make a rivet far superior to the old hand-driven ones. They can be readily moved into any position, are operated by one man, and can reach many places where the old system of hand riveting was ineffective.

These hammers may also be used for chipping or caulking by putting in the required tool. No tool is shown in the illustration, and to be used for riveting a cup must be introduced.

Rivets are made of soft steel and of such lengths that, when put into the holes, enough metal will project, so that when compressed it will completely fill the hole and still enough remain to form the head. Before being put into the holes, they should be heated to a cherry-red. Great care must be taken that the metal shall not be burned in the heating, as otherwise it will be hard and brittle. Rivets should also not be driven when too cold, or they will not completely fill the hole, and the heads, though apparently all right, will be weak. The heads should

Fig. 23. Pneumatic Riveter.

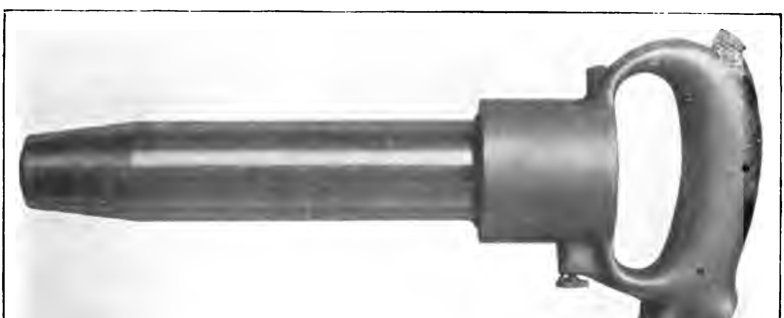


Fig. 24. Pneumatic Hammer.

be smooth and free from cracks, and should be concentric with their holes.

The work having now been made solid, those pieces which in the finished structure are to have butting joints, such as stringers and chord sections, must now be made of exact lengths. It has been seen that all pieces to be machined were first cut $\frac{1}{4}$ inch longer than the finished dimensions. The work of bringing them to exact length is called facing, and is done by a machine called a rotary planer. Such a machine, made by Bement Miles & Company of Philadelphia, is shown in Fig. 25.

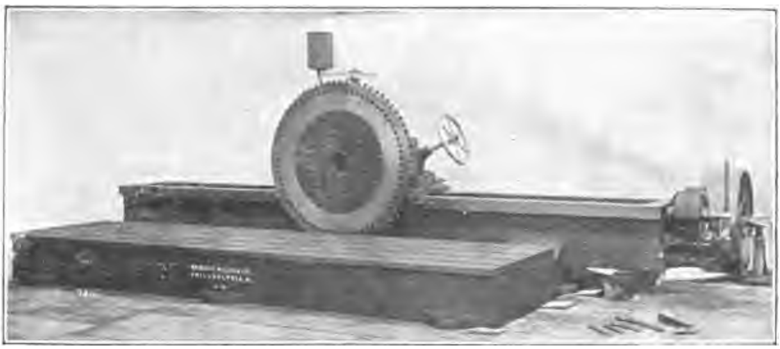


Fig. 25. Rotary Planer.

The piece to be faced is clamped into position on the bed in front of the disc which carries the cutting tools in the holes near its circumference. The edges of these tools are all in the same plane, and the disc is turned, while at the same time it moves slowly forward. Any metal on the piece which projects beyond the plane of the cutting tools is therefore chipped away, and so, in several successive operations if need be, all parts of its end are made to lie in a true plane.

The machine shown is not mounted upon a turntable, but this is the usual custom, so as to avoid slewing long members across the shop whenever a skew face is to be made.

Those members that belong to pin-connected structures are now taken to the drill presses, where the pin holes are carefully bored in exact position.

All of the work in the shops has now been completed, and the various members, before being taken to the shipping yard, are carefully weighed and inspected, and a record kept, both for the purpose of knowing the capacity of the shop, and also exactly what work has been completed.

The equipment of the assembling shop consists of ample skids, on which the work can be brought together, the stationary and portable drills and reamers, the pneumatic and hydraulic riveters, the planing machines and drill presses, and the overhead cranes, reaching to every part of the floor space. Numerous small tracks also permit of the moving of any piece from either end or to any part of the shop; and, finally, there are the scales on which the material is weighed before it is passed on to be painted and prepared for shipment.

ART. 23. THE MACHINE SHOP.

While a very small percentage of all the work done by a bridge company is machined work proper, yet, for reasons of internal economy, this shop is perhaps the most important of the plant. It should be so equipped as to be able to replace or repair, promptly, practically every part of any machine in the plant. The almost endless variety of such repair work renders necessary the constant employment of a much larger force than would be required for the bridge manufacture alone.

The parts of steel structures which have machine work done upon them are pins, rollers, bed and sole plates, and anchor bolts. The various bearing parts of turntables are also made here.

The machinery in this shop consists of a variety of drills, planers, and lathes of various sizes. The material for the pins and rollers usually comes from the mills, and has a diameter $\frac{1}{8}$ inch greater than that of the finished pin. It is turned down to the required size and the threads cut on the ends for the pin nuts. The planers are used to smooth off the faces of the bed and sole plates for the expansion joints.

Connected with the machine shop there is a storehouse, where is kept a large assortment of bolts of all sizes and lengths up to 1-inch diameter. Larger bolts than this are special, and are made by the shop as required. All steel and iron castings which require facing are finished up here. These castings are generally purchased from some outside company, there being few bridge companies who operate their own foundries.

ART. 24. THE FORGE SHOP.

The three general classes of work done in this shop are the manufacture of rivets, the general smith work required on a steel structure, and the making of eye-bars.

The soft steel rods from which the rivets are made are heated in a furnace and then passed into the rivet-making machine, which, at one operation, upsets the rod, makes the rivet head, and cuts off the proper length from the rod. The rivet thus formed drops from the machine, and when cool is placed in stock. The rivets most generally used in structural work are $\frac{3}{4}$ and $\frac{7}{8}$ inch in diameter.

Special bolts are made in the same way in a similar machine, and then have their threads cut in the machine shop.

The general smith work consists of the bending of plates, angles, and shapes, as called for by the drawings, the making of clevises and loop eye-rods, and the upsetting of the ends of adjustable tension members. All forgings that may be needed

by the plant, such as punches, drift pins, and riveting cups, are also made here.

In the eye-bar department of this shop all of the large pure tension members of pin-connected structures are made. The standard dimensions of eye-bar heads may be found in any of the handbooks where they are given for various diameters of pin in bars of varying width. Specifications always require that the heads of the bars shall be so proportioned that, if tested to destruction, the break will occur in the body of the bar rather than in any part of the head or neck. The form of head is, however, not specified, but left subject to the approval of the engineer in charge of the work. The excess of area in the head over that in the bar is from thirty-three to fifty percent, so as to insure ample strength in that part of the bar.

The steel flats from which the eye-bars are made have about four feet of one of their ends heated to a bright cherry-red in a forge. This heated end is then placed in the upsetting machine and a solid head formed, which is almost exactly the size of the finished head. After reheating, it is taken to the steam hammer, placed in the die, and hammered out so as to thoroughly solidify the material and be made of the proper size and thickness. Before leaving the hammer a hole is punched in the center of the head to facilitate the boring of the finished hole. During the entire process a careful watch is kept for flaws, and all discovered must be cut out, for a fault in a steel bar will never weld perfectly, and a flaw, no matter how small, is always a source of weakness; the head on the other end of the bar is made in the same way. A number of the bars are now taken to the annealing furnace, in which they are piled on edge, but not in contact. The temperature of the furnace is gradually raised with wood or gas fuel until the bars become of a cherry-red, when the fire is slowly allowed to go out and the bars to gradually cool. The object of this process is to permit

the metal in the bars to arrange itself naturally and so to remove any local stresses which might exist. Such stresses may be caused either in upsetting or in forging the head.

When cool enough to handle, the eye-bars are taken to the straightening press and are made true to line; they then pass on to the boring mill, where the pin holes are finally made. COOPER's Specifications, 1901, require that "chord pins shall fit the pin holes within $\frac{1}{50}$ of an inch for pins less than $4\frac{1}{2}$ inches diameter; for pins of a larger diameter the clearance may be $\frac{1}{32}$ inch," and also that "bars which are to be placed side by side in the structure shall be bored at the same temperature and of such equal length that upon being piled on each other the pins shall pass through the holes at both ends without driving." The same specifications also require that "the bars must be bored to lengths not varying from the calculated lengths more than $\frac{1}{64}$ of an inch for each 25 feet of total length."

The finished bars should be smooth, straight, and free from flaws. The pin holes should be at the center of the heads and on the center line of the bar.

The forge shop buildings should be thoroughly ventilated and special apparatus provided for removing the gases and smoke which arise from the furnaces and forges. To do this in the most efficient and economical way a system of power ventilation must be installed, the fresh air being forced into the shop near the floor, and the gases removed at the ventilators on the ridge of the building.

ART. 25. INSPECTION.

After the material has left the shops, and before it is prepared for shipment, it must be gone over carefully and in detail in order to see that it conforms in every way to the specifications

and to the drawings. This work is done by the inspectors, both the bridge company and the purchaser having men at the shops for this purpose.

The company's inspector has to look over all work which comes from the shops, and his approval is necessary before any piece is shipped by the company. He usually has a number of assistants and is principally concerned in seeing that the work will fit together easily and accurately in the field.

The purchaser's inspector is employed by those for whom the material is being made, and is their representative on the ground. He is concerned only with his own work, and when the structure is to be erected by the bridge company he need be very particular only regarding the quality of the material and the character of the workmanship. When, however, his own company is to erect the structure, he must examine carefully into every detail and be perfectly satisfied before the material is shipped that the different members will fit together. In this latter case his acceptance is final, and should any errors creep through, his company must bear the expense of correcting them. No material can be shipped until he has stamped or marked it and thus signified his acceptance.

A large percentage of all inspection done is made by inspection companies or firms. These companies bid for the work of inspecting a lot of material at a ton price. They employ a large number of inspectors, whom they send from place to place to look over the work which they have in hand. At a large bridge concern they usually have a permanent man who inspects all of their work passing through that plant. In this way the inspectors can be kept constantly busy, and the cost of inspection to the purchaser will, in many cases, be greatly reduced. The principal objection to this inspection — and it is one held by many purchasers — is that the actual inspector is not personally known,

and also, though the inspection firm may be well and widely known, it will often not hold itself financially responsible for the work of its employees. The good work done by these firms is, however, amply attested by the volume of their business and their professional reputation.

The purchaser's inspector, when he arrives at the plant where the bridges to be inspected are being manufactured, should at once begin to familiarize himself with the shops, the methods by which the work is done, and also become acquainted both with the officers of the company and the foremen having actual charge of the work. He will be furnished by the company with plans and drawings of the work, and with all other information that he may need.

He should ascertain where the material which is to go into his bridges will be obtained, and then be on hand at the mills when it is being rolled. Before the material is taken from the mills to the bridge plant, it should be carefully examined for flaws and surface defects, and the results of the tests made on specimens cut from it be studied. The material being accepted, it should be stamped by the inspector with his hammer, and the position of the stamp indicated by a circle of paint. This stamp means that the material may now be sent on and be used in manufacturing the bridge members. Any material not so marked is not allowed to be forwarded.

At the bridge plant the inspector should keep a general oversight of the work as it is passing through the shops. He should watch the making of the templets, the punching of the holes, the shearing of the plates, the assembling and the riveting, all with a view to seeing that the material is in no way mistreated or injured. He can insist on no special methods of manufacture or procedure unless they be called for by the specifications, but he should see that all work is well and properly done, and that no

bad work which might weaken the structure shall be covered up so that the final inspection would miss it.

The work having passed out of the shops, he now proceeds to examine it critically, with the view of finding all defects that may exist. With his hammer he taps the rivet heads and marks all such as may be loose or defective to be cut out and replaced. He looks to see that all clearances are correct, that the proper sizes of material have been used in building up the members, that no holes have been left open, and that no field-rivet holes have been closed. He also looks out for all total and detail dimensions, and sees that the members conform to the requirements of the specifications and drawings.

He reports all errors he may find, or changes which may seem necessary, to the company's inspector, who issues an order to the shop to have them corrected. When this is done he stamps each piece with his hammer, as an attest that it has been accepted by him and is now ready to be prepared for shipment. He then also sees that the painting is properly done, that all small pieces are properly boxed, and finally that all parts of the structure are promptly shipped to their destination.

In general, while the work is in progress, the purchaser's inspector should at all times be on hand, ready to settle all points as they arise. He should be slow to decide a question, and never render rash or hasty opinions; he should temper his judgment with discretion, for only in this way can he command the respect of the company's employees, who naturally look upon him as being more or less of an interloper, and who, if antagonized, can easily make life a burden for him. He should remember always that he is in a judicial capacity, and when pronouncing on the evidence before him should be just, equally to himself, to those whom he represents, and to those whose work he is judging.

ART. 26. PAINTING AND SHIPMENT.

The material having been inspected and stamped, it is now ready to be painted and finally prepared for shipment.

There are in the market hundreds of varieties of paints of different kinds, mineral paints, asphaltum paints, and graphite paints. Many of them are good and many more are not. The paint to be used on any structure is generally called for in the specifications, but the standard shop painting and that generally required is one coat of boiled linseed oil. This oil in drying makes a good base for the field painting, which is done after erection.

Before painting, the work should be thoroughly cleaned of all shop oil, rust, and scale, so that the paint will come into direct contact with the metal. Some specifications, but rarely, and only on important work, require that all surfaces before painting shall be cleaned with a sand blast so as to assure a perfectly clean metal face.

The commonest and best method of applying the paint is by hand, with a brush. In some plants, however, it is done by forcing compressed air through the paint and blowing it in a spray against the metal. The only advantage of this method is its rapidity. Unless very carefully done, it will not distribute the paint evenly, and then it becomes an expensive method on account of the great quantity of paint wasted.

After being painted the work should be marked with its job number, which is the shop number of the structure, and also with its erection marks, which are the letters and numbers given on the drawings to identify the separate pieces. These marks should be put on clearly and conspicuously, so that no time will be lost in the field in picking out the various members. All work which is to be carried to a distance, and especially export work, should have its marks painted, and also stamped

with steel stamps, as the paint marks are easily obliterated or rendered indistinguishable by frequent handlings. When large export shipments are made, the expedient of painting all parts of the same structure the same color is often resorted to, in order to facilitate the sorting and distribution at the point where the shipment is unloaded.

All of the small parts of the structure, such as rivets, bolts, pins, roller bearings, etc., are carefully packed in boxes and each box marked with the job number as well as with a list of its contents.

Before being shipped the different parts are again carefully weighed, as the price paid is often per pound of finished material. The inspector must also compute carefully the weight of each piece and compare it with the scale weight. A variation of 2½ per cent between the estimated and scale weights is usually allowed. In case the difference should be greater than this, an examination must be made and the piece reweighed if necessary.

The bridge company's work is now complete unless it is to erect the structure. The different parts are all loaded on cars and forwarded to their destination. Shipping bills are made out and sent to the purchaser; copies of them are also furnished to the inspector. These bills give the name of each piece, its package number, its gross weight, net weight, and tare, and show also the total number of pieces in the shipment, as well as its total weight. The inspector now examines these bills, and, if correct, approves and sends them to his people, together with his final report on the inspection of the structure.

ART. 27. TESTS AND TESTING.

The ordinary tests made on material are on small specimens and are for the purpose of determining its quality. They are made on testing machines, which usually have a capacity of

about 100 000 pounds in either tension or compression. The specimen to be tested is cut directly from some plate or shape and smoothed or turned down. Its breadth and thickness are carefully noted, and a length of 8 inches laid off on it in inch divisions. It is then put into the machine and broken, and the yield point and ultimate strength noted. After breaking, the breadth and thickness of the specimen at the point of rupture are noted, as also the elongation in some specified length — usually 8 inches. From these data the yield point and ultimate strength in pounds per square inch are computed, as also the reduced area, the percentage of elongation, and the percentage reduction of area. The characteristics of the fracture are also noted as having a bearing on the quality of the material as respects toughness and brittleness.

Two specimens are taken from each heat, one to be pulled in tension, and the other for the "quench test," which consists in heating the specimen to a dull cherry-red, cooling in water, and then bending cold. COOPER's Specifications, 1901, require that this specimen "must stand bending to a curve whose inner radius is one and a half times the thickness of the sample without cracking." Medium steel for bridge structures is usually required to have an ultimate strength of 60 000 to 68 000 pounds per square inch, an elastic limit of not less than one-half of the ultimate strength, and a minimum ultimate elongation of 22 percent determined from a gaged length of 8 inches.

Bridge companies have their own testing machines, which are always at the disposal of the inspector for his work. Most of this testing is, however, done at the mills, where there are large laboratories devoted entirely to such work.

Specifications occasionally call for tests on full-sized members of a structure, though this is sometimes left to the discretion of the engineer in charge of the work. Such testing requires

large machines of great capacity, and the following is a list of the largest machines of this kind in the United States; they may be used in compression as readily as in tension.

The machine of the Phœnix Iron Company at Phœnixville, Pa., is the largest of its kind in the world. It can exert a stress of 2 160 000 pounds, and can take a specimen 45 feet long with a stretch of 15 percent. For a description of this machine, see Engineering News, Dec. 28, 1893.

The machines of the American Bridge Company at Athens, Pa., Edgemore, Del., and Pittsburg, Pa., have capacities respectively of 1 244 000, 700 000, and 600 000 pounds, while the machine belonging to the United States Government at the Watertown Arsenal has a capacity of 1 000 000 pounds. This machine is remarkable for its delicacy and precision. It can break a hair and a bar of steel 30 feet long, and the relative error in measuring the ultimate strength will be closely equal in the two cases.

ART. 28. THE ORGANIZATION OF A BRIDGE COMPANY.

The rapid development of, and the fierce competition in, bridge and structural steel business have made it necessary that a bridge company, in order to be successful, must have an organization as complex and yet as nice in its every detail as that of an army which has been in the field for many years. This, in fact, is true of every great business, for it is almost a fundamental law that the company having the best organization will also have the greatest profits. This is not the place for a discussion of all of the intricacies of any business concern, but it will be well to briefly indicate to the student just what the word "organization" means.

A bridge company takes contracts for and delivers to the purchaser steel structures of every kind. It may take the con-

tract for an entire structure or only for the steel work. In general, on nearly every contract there is some work which the company cannot do, and this is sublet to some one else. The sales department of the company looks out for all of this portion of the business. It has agents in all parts of the country who are always on the lookout for new contracts, and it also sublets any work which it cannot do but often finds it advisable to take in order to obtain advantages over its competitors. On the energy, watchfulness, and efficiency of this department depends very largely the profit of the company; if it can get but little work, its earnings will be small.

After a contract is secured it is passed on to the engineering department, which in the first place made the preliminary estimates for the sales department, and now it takes up the work of the detail design in its offices and drafting room. The plans being completed, they are sent into the shops, but the shops cannot go ahead without material to work upon, and this end of the business is looked after by the purchasing department. This department makes contracts for and obtains all materials needed by the plant, — everything that is required for building the structures under contract as well as everything necessary in the operation of the entire plant, whether it be a paper of pins for office use or a new dynamo for generating power.

Then there is the operating department, which concerns itself with the progress of the work and the internal economy of the shops. It must see that every machine is kept running just as constantly as possible, that templets are used over again whereever practicable, that the greatest duty is gotten out of the engines and boilers, that no unnecessary men are employed, that all proper facilities for the work are furnished, and above all that the shops shall turn out the greatest possible tonnage, for this is the real measure of a plant's earning capacity. When the material is out of the shops, the engineering department

again takes a hand in the inspection, and pronounces on the work of the operating department.

The shipping department now loads the material, arranges for freight rates and routes, and notifies the purchaser of the shipment. Then, when the material arrives at the site where the structure is to be put up, the erecting department takes charge of it and puts it into place. And finally there is the financial department, which sends out the bills for work done and contracts completed, and receives payment for the same.

Besides the departments already mentioned there is a pay department, which looks after and keeps the time of every man in the company's employ, and disburses the wages on pay day. Each of these departments may be subdivided, but it must at all times be in readiness to answer any questions which may be asked by the general superintendent, who must keep in touch with all work in the plant, and also with the progress and condition of each department. In order to secure the most satisfactory results, namely, the best bridges and the highest profits, it is necessary that all these separate departments shall work smoothly and together, each helping the other to attain the desired ends.

As an illustration of the thoroughness with which the accounts are kept, the following classification of shop processes used in some bridge plants is given:

1. Unloading.
2. Running In.
3. Templets.
4. Straightening.
5. Edging.
6. Shearing.
7. Laying Out.
8. Punching.
9. Coping.
10. Rotary Planing.
11. Fitting.
12. Bending.
13. Hand Riveting.
14. Solid Drilling.
15. Reaming.
16. Riveting.
17. Chipping.
18. Assembling and Reaming.
19. Shop Handling.
20. Boring.
21. Annealing.
22. Upsetting and Forging.
23. Turning.
24. Planing.
25. Machine Work.
26. Grinding.
27. Running Out.
28. Inspecting.
29. Painting.
30. Loading.

At the close of each day each man employed in the shops fills out a time card, headed with the shop number of the job, and then gives the number of hours spent in each of the above operations, specifying it by the number prefixed. It is thus possible to determine for each bridge the cost of each and all of these thirty items. If in any case the relative cost of any item much exceeds its proper percentage, an investigation may be made and measures taken that such excess shall not again occur. If, however, the cost of any item comes out materially less than the usual percentage, the foreman in charge should be complimented and encouraged to maintain a similar record on future jobs.

The cost of the product to the purchaser is the cost of production plus the profit of the bridge plant. A careful account is kept of each department, and each charged with its proper share. The different charges which are usually made are to management, to engineering, to shops, to sales, to shipping, to erection, and to profit, the latter including interest charges, depreciation of plant, contingency and sinking funds.

The above brief outline will serve to show the student that while the successful conduct of a bridge business is primarily dependent on the engineering department, which is responsible for the strength and safety of the structures built by the company, yet that there are also other and equally important departments which require just as earnest work and even higher executive capacity. The man thoroughly trained in the engineering work is the one best fitted to take up the more trying and responsible positions in the other departments.

CHAPTER V.

TABLES AND STANDARDS.

ART. 29. MANUFACTURERS' HANDBOOKS.

The student of bridge design will find it absolutely necessary to have at hand one of the handbooks issued by the manufacturers of structural materials. There are a number of these, the best known being those popularly called the Phœnix, Carnegie, Pencoyd, Jones and Laughlins, Passaic, and the Cambria handbooks. The titles of these books are as follows: Useful Information for Architects, Engineers, etc., issued by the Phœnix Iron Works, Phœnixville, Pa.; Pocket Companion, issued by the Carnegie Steel Company, Pittsburg, Pa.; Steel in Construction, edited by James Christie, and issued by A. and P. Roberts Company, Philadelphia, and the American Bridge Company, New York; Standard Steel Construction, issued by Jones and Laughlins, Limited, Pittsburg, Pa.; Structural Steel and Iron, edited by G. H. Blakeley, and issued by the Passaic Rolling Mill Company, Paterson, N. J.; Cambria Steel, issued by the Cambria Steel Company, Johnstown, Pa. The Pencoyd, Cambria, and Carnegie handbooks are the most complete and in general use for work in bridge design. The Passaic and the Jones and Laughlins handbooks are mainly adapted for the design of steel buildings, but are also used in bridge design. The prices of these books range from one to two dollars, but they can usually be obtained in quantities at a special rate by college students.

The handbooks contain full tables of all the market shapes of steel manufactured by the respective firms, stating weights, areas of sections, positions of centers of gravity, moments of

inertia, radii of gyration, and other constants. Tables are also given for the shearing and bearing values of rivets, the bearing values of pin plates, the resisting moments of pins, standard bolts, eye-bars, bridge pins, and other details, as well as for the weight and strength of materials used in bridge and building construction. The necessary computations in bridge design may be greatly shortened by the use of these tables. In the following pages a few tables are presented which are more complete than those in the handbooks.

In order to obtain uniformity in the work done at its various plants, the American Bridge Company issued, in 1901, a book entitled Standards for Structural Details. It contains a number of tables similar to those in the handbooks as well as some new ones, together with details relating to the use of corrugated steel for roofing and siding, and of standard doors and windows. The rules for making shop drawings are referred to in Art. 17, and some of them are reprinted.

OSBORN's Tables of Moments of Inertia and Squares of the Radii of Gyration economize time in designing the struts in lateral and sway bracing, and the posts and upper chords of trusses. BUCHANAN's, SMOLEY's, or HALL's Tables of Squares are useful in finding the lengths of diagonal members.

ART. 30. GENERAL SPECIFICATIONS.

A number of general specifications for steel railroad bridges and viaducts have been prepared by consulting engineers, and are published for general use, including those of COOPER, WADDELL, THE OSBORN COMPANY, THACHER, BOUSCAREN, SEAMAN, and SCHAUB. COOPER's Specifications have been in use for many years, and probably more bridges have been built in accordance with them than with any other set. WADDELL's Specifications are much more elaborate and explicit than the

others, and are particularly serviceable to students, since they embody recommendations based on experience in regard to a considerable number of details whose determination is not wholly subject to theory and to which no reference is usually made.

Many of the leading railroads have their own standard specifications, the most noted being those of the Pennsylvania, New York Central, Baltimore and Ohio, Norfolk and Western, Union Pacific, Southern Pacific, and the Chicago, Rock Island, and Pacific railroads.

Some of the manufacturers of bridges also issue general specifications, the principal ones now available being those of the American Bridge Company and of the Phœnix Bridge Company.

These specifications usually indicate the types of bridges for different spans, the clearance required for the trains, the character of the wooden floor, the dead, live, wind, and traction loads, allowance for impact and vibration (in some cases), and the safe unit stresses. They also give the general limits in designing, including the minimum thickness of material and sizes of shapes, the general principles in designing structures, the details of riveting, the details of design and construction of beam, plate girder, riveted girder, and pin-connected spans, as well as of viaducts, the quality of material and workmanship, inspection, painting, erection, and final test.

A study of the provisions of the specifications adopted for any given design is a necessary preliminary to the computations and drawing involved in making the design. It will greatly facilitate the student's work to be provided with an index referring to the various paragraphs to be consulted in designing the different parts of the bridge, so that all of them may be duly considered at the proper time.

A comparison of the specifications referred to above will show to the student marked differences in the allowable unit stresses prescribed. One of the principal reasons for this lies in the fact that some make the allowances for impact and vibration by increasing the live load stresses by a percentage, which may be either fixed or variable, while in others these allowances are made by modifying the safe unit stresses. The differences relating to many other details of design and construction will be referred to in connection with the designs in Chapters VII and IX.

General specifications for steel highway bridges have also been prepared by the first four consulting engineers mentioned at the beginning of this article, those of WADDELL and COOPER having been very extensively employed. Many of the manufacturers of bridges also issue such specifications, those of the American Bridge Company being the most complete of that class. Some of the standard specifications for railroad bridges adopted by the railroads also contain provisions relating to highway bridges, as well as to roofs and buildings. FOWLER'S general specifications relate exclusively to steel roofs and buildings.

Most of the specifications for railroad and highway bridges contain an appendix consisting of tables of maximum moments and shears, coefficients of impact, equivalent uniform loads, permissible unit stresses for columns, and other useful data. The specifications of the New York Central and Hudson River Railroad contain plates showing standard details for beam, plate girder, and riveted truss bridges. Those relating to plate girders are reproduced in Figs. 13, 14, 15, and 23, and those for riveted bridges are given on Plate VII.

The following references contain important discussions of bridge specifications:

Working Stresses for Railroad Bridges. Editorial. Railroad Gazette, vol. 30, page 797, Nov. 4, 1898.

The Launhardt Formula, and Railroad Bridge Specifications. By HENRY B. SEAMAN. Transactions American Society of Civil Engineers, vol. 41, page 140, June, 1899.

The Determination of the Safe Working Stress for Railway Bridges of Wrought Iron and Steel. By E. HERBERT STONE. Trans. Am. Soc. C. E., vol. 41, page 467, June, 1899.

Proposed Specifications for Steel Railway Bridges. By J. W. SCHAUB. Journal Western Society of Engineers, vol. 5, page 347, Oct., 1900.

Notes on Specifications for Bridge and Structural Steel. By P. S. HILDRETH. Railroad Gazette, vol. 33, p. 517, July 19, 1901.

Highway Bridge Design and Construction. Editorial. Engineering Record, vol. 44, page 217, Sept. 7, 1901.

On Specifications for the Strength of Iron Bridges. By JOSEPH M. WILSON. Trans. Am. Soc. C. E., vol. 15, page 389, June, 1886. Although the specifications given in this paper have been superseded by later ones, the discussion still contains much useful material for the student.

The following articles also contain the results of tests and discussions relating to safe unit stresses:

What is the Life of an Iron Railroad Bridge? By J. E. GREINER. Trans. Am. Soc. C. E., vol. 34, page 294, Oct., 1895.

The Condition of Steel in Bridge Pins. By A. C. CUNNINGHAM, Trans. Am. Soc. C. E., vol. 36, page 91, Dec., 1896.

The following reference gives the revised specifications for structural steel adopted by the American Section of the International Association of Testing Materials: Proposed Standard Specifications for Steel for Bridges, Ships, Forgings, etc. Engineering News, vol. 46, page 11, July 4, 1901.

ART. 31. LIVE LOADS FOR HIGHWAY BRIDGES.

The specifications referred to in Art. 30 give the loads to be used in designing bridge structures which shall have sufficient strength under the various conditions indicated. Such loads, of course, are supposed to closely represent the maximum weights to which the structure is liable.

The extensive use of road rollers, traction engines, electric cars, and other vehicles carrying heavy loads, requires the specification of concentrated loads in designing highway bridges in addition to the uniformly distributed loads, which are supposed to represent the weight of a crowd of people.

According to the specifications of the American Bridge Company, highway bridges are divided into six classes, viz.:

Class A. — For city traffic.

Class B. — For suburban or interurban traffic with heavy electric cars.

Class C. — For country roads with light electric cars or heavy highway traffic.

Class D. — For country roads with ordinary highway traffic.

Class E 1. — For heavy electric street railways only.

Class E 2. — For light electric street railways only.

In designing the floor and its supports, a concentrated live lead on two axles, 10 feet between centers, and assumed to occupy a width of 12 feet, is to be placed on each street car track of 5 feet gage, this load being 24 tons for classes A and B, and 18 tons for class C; or a concentrated load having the same distribution, width, and gage, is to be placed on any part of the roadway, the load being 24 tons for class A, and 12 tons for classes B and C. Upon the remaining portion of the floor, including sidewalks, there is to be placed a uniformly distributed load of 100 pounds per square foot for classes A, B, and C.

For the floor and its supports of class D, the load shall be either a concentrated load of 6 tons distributed as for the other classes, or 80 pounds per square foot of total floor surface.

In designing the trusses for spans up to 100 feet, the live load per linear foot of car track and assumed to occupy a width of 12 feet, is to be 1800 pounds for classes A and B, and 1200 pounds for class C, while that upon the remaining floor surface is to be 100 pounds per square foot for class A, and 80 pounds per square foot for classes B and C. For spans of 200 feet or more the load per linear foot of track is 1200 pounds for classes A and B, and 1000 pounds for class C, while the load on the remaining floor surface is 80 pounds per square foot for class A, and 60 pounds per square foot for classes B and C. For the trusses of class D the live load is 80 pounds per square foot of total floor surface for spans up to 75 feet, and 55 pounds for spans of 200 feet and over. For intermediate spans the loads are in all cases to be reduced proportionally from the higher to the lower values.

The bridges of class E 1 are designed for those loads which relate to the car tracks only in class A or B, while the bridges of class E 2 are designed for the corresponding loads in class C.

This classification of highway bridges and the corresponding live loads are substantially the same as those in COOPER's specifications, which were published a few months before.

The loads specified by WADDELL differ considerably from the above. The concentrated loads are distributed to three wheels in the case of the road roller, and to four wheels in the other cases. The weights and spacing of wheels are given for several classes of electric cars, together with the equivalent uniform loads. The uniformly distributed load is determined by means of a diagram, the length of the load on the span being considered, which causes the maximum stress in any given member of the truss or floor. In the case of bridges with exterior

sidewalks it is also stated what portion of the roadway and sidewalks shall be loaded for finding the stresses in the trusses and the floor beams.

ART. 32. LIVE LOADS FOR RAILROAD BRIDGES.

For railroad bridges the "compromise standard system" of live loads recommended by WADDELL in 1893, after a discussion of the subject by many engineers, are given here as a standard which will probably continue to be extensively adopted. The typical consolidation locomotives are divided into ten classes,

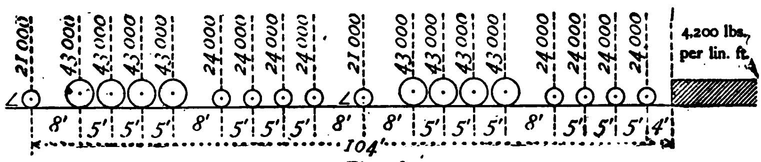


Fig. 26.

which, together with the loads on the axles, are given in the table. Fig. 26 shows the spacing of the wheels or axles, this being the same for all the classes, and the loads for class T. The distance from the front pilot wheel to the beginning of the uniform load is in all cases 104 feet, and the loads are stated in pounds.

CLASS.	Load on Pilot Wheel Axle.	Load on Each Driving Wheel Axle.	Load on Each Tender Wheel Axle.	Total Weight of One Locomotive and Tender.	Uniform Load per Linear Foot of Track.
Q	24 000	52 000	27 000	340 000	4800
R	23 000	49 000	26 000	323 000	4600
S	22 000	46 000	25 000	306 000	4400
T	21 000	43 000	24 000	389 000	4200
U	20 000	40 000	23 000	272 000	4000
V	19 000	37 000	22 000	255 000	3800
W	18 000	34 000	21 000	238 000	3600
X	17 000	31 000	20 000	221 000	3400
Y	16 000	28 000	19 000	204 000	3200
Z	15 000	25 000	18 000	187 000	3000

Alternative loads on two axles 7 feet apart are also given for each class, the load on both rails of a single track for each axle being as follows: 58 000 pounds for class Q, 56 000 for R, 54 000 for S, 52 000 for T, 50 000 for U, 48 000 for V, 46 000 for W, 44 000 for X, 42 000 for Y, and 40 000 for Z. These loads represent the heavier drivers of the typical passenger locomotives, and apply only to beams of very short spans and to cross-ties in the track on bridges. The specifications also contain diagrams of end shears for spans from 7 to 100 feet, and of equivalent uniform loads for spans from 10 to 500 feet for the ten classes of live loads in the compromise standard system.

COOPER's standard train loading, class E 50, is shown in Fig. 27. The spacing is the same for all the classes, and the loads on the wheels are arranged so that the corresponding loads of any two classes have the same ratio. For example, all the loads

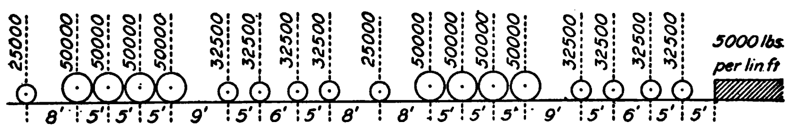


Fig. 27.

of class E 40 are four-fifths of the loads of class E 50. Any stresses, moments, or shears due to the loading of one class may thus be readily derived from those obtained for the loading of another class by applying the ratio of their class numbers.

A table comparing the weights of recent heavy passenger and freight locomotives is given in the appendix to COOPER's Specifications, and in Engineering Record, vol. 44, page 468, Nov. 16, 1901, may be seen diagrams comparing the equivalent uniform loads of a number of these locomotives for short and long spans, as well as the shears produced by them in spans from 10 to 120 feet.

ART. 33. RIVET PROPORTIONS.

The rivet proportions given in the following table are the Pencoyd standard adopted by the American Bridge Company. For finished or button heads the diameter equals $1\frac{1}{2}$ times the diameter of the shank plus one-eighth of an inch, and the height equals 0.425 times the diameter of the head. For countersunk

SIZE.	FINISHED HEAD			COUNTERSUNK.	
Diameter.	Height.	Diameter.	Radius.	Depth.	Diameter.
$\frac{3}{8}$	$\frac{19}{64}$	$\frac{11}{16}$	$\frac{7}{16}$	$\frac{3}{16}$	$\frac{19}{32}$
$\frac{1}{2}$	$\frac{3}{8}$	$\frac{7}{8}$	$\frac{9}{16}$	$\frac{1}{4}$	$\frac{25}{32}$
$\frac{5}{8}$	$\frac{29}{64}$	$1\frac{1}{16}$	$\frac{43}{64}$	$\frac{5}{16}$	1
$\frac{3}{4}$	$\frac{17}{32}$	$1\frac{1}{4}$	$\frac{51}{64}$	$\frac{3}{8}$	$1\frac{3}{16}$
$\frac{7}{8}$	$\frac{39}{64}$	$1\frac{7}{16}$	$\frac{59}{64}$	$\frac{7}{16}$	$1\frac{3}{8}$
1	$\frac{11}{16}$	$1\frac{5}{8}$	$1\frac{1}{32}$	$\frac{1}{2}$	$1\frac{9}{16}$

(All dimensions are in inches.)

rivets the depth of head equals one-half of the diameter of the shank, the bevel of the head being 60 degrees. Since the finished head of a rivet is not quite a hemisphere, the diameter of its base is a little less than twice the radius of its spherical surface.

According to the Cambria standard, the height of the finished head equals six-tenths of the diameter of the shank, while the radius of the head equals three-fourths of the diameter of the shank plus one-sixteenth of an inch. The diameter of the countersunk head is made the same as that of the button head. This standard also recommends that in figuring clearances for rivet heads, a height of $\frac{5}{8}$ inch should be allowed for $\frac{3}{4}$-inch rivets, and of $\frac{3}{4}$ inch for $\frac{7}{8}$-inch rivets.

The clearance between any rivet head and an adjacent surface or projection must allow room for the riveting tool, — at least $\frac{3}{8}$-inch clearance is required. For a $\frac{7}{8}$-inch rivet the distance from

the center of the rivet to the back of an adjacent angle should not be less than $1\frac{1}{4}$ inches, while for a $\frac{3}{4}$-inch rivet it should not be less than $1\frac{1}{8}$ inches. The minimum distance between the center of a rivet in one leg of an angle and the projection of a rivet head located on the other leg is $1\frac{1}{8}$ inches. This clearance determines the minimum stagger.

When stiffeners are crimped over the flange angles, the distance from the edge of the flange angle to the next rivet in the stiffener should be $1\frac{1}{2}$ inches plus twice the thickness of the flange angle, but never less than 2 inches. The distance from the center of a rivet to the edge of a plate should not be less than $1\frac{1}{2}$ diameters; and whenever practicable, it should be at least 2 diameters, and it should not exceed 8 times the thickness of the plate.

The maximum pitch of rivets in the direction of the stress should not exceed 6 inches nor 16 times the thickness of the thinnest outside plate, and the minimum pitch should not be less than 3 diameters of the rivet. Additional requirements relating to the pitch and location of rivets will be given in connection with the designs in the following chapters. Rivets should not be countersunk in plates whose thickness is less than the depth of the countersunk head, and it is preferable that the plate should have some bearing in the shank of the rivet.

The rivets chiefly used in bridge work are $\frac{7}{8}$ inch in diameter, but $\frac{3}{4}$-inch and $\frac{5}{8}$-inch rivets are employed in lacing or in other minor details. In some very heavy work rivets 1 inch in diameter are being used. Rivet tests show that the grip length should not exceed 5 diameters for machine-driven rivets. See Engineering News, vol. 24, page 500, December 6, 1890.

ART. 34. RIVET SPACING IN ANGLES.

The following table gives the standard spacing adopted by the American Bridge Company, together with the maximum

diameters of the rivets to be used. As shown in Fig. 28, *l* represents the length of the angle leg, *a* the distance from the corner to the pitch line of a single row of rivets, this distance being known as the gage, *b* the corresponding distance to the first row for double riveting, and *c* the distance between the pitch lines of the two rows. Two values of *b* and *c* are given for the 6-inch angle, the first being used when the thickness of the angle does not exceed ⅜ inch.

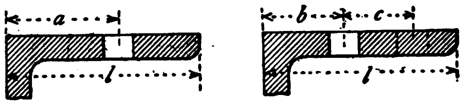


Fig. 28.

LENGTH OF ANGLE LEG *l*	SPACING			MAXIMUM DIAMETER OF RIVET.
	a	*b*	*c*	
8	4½	3	3	⅞
7	4	2½	3	⅞
6	3½	{ 2¼ / 2½	{ 2½ / 2¼	⅞
5	3	2	1¾	⅞
4	2¼			⅞
3½	2			⅞
3	1¾			⅞
2¼	1⅝			¾
2½	1⅜			⅝
2¼	1¼			⅝
2	1⅛			½

(All dimensions are in inches.)

In several references to this article in succeeding chapters the statement is made that the 3½-inch angles are the smallest in which ⅞-inch rivets may be used. The designs given in those chapters were made before the revised standards were received, and are in accordance with the former Pencoyd standard in this respect.

ART. 35. PIN PLATE AND RIVET DIAGRAM.

The diagram in Fig. 29 is constructed for the unit stresses and diameters of rivets there given. The diameters of the pins are laid off as abscissas, and the bearing values for the pins as ordinates, the linear bearing on the pins being marked on the lines radiating from the lower left-hand corner. The allowable stress for an 8-inch pin with a bearing of $1\frac{7}{8}$ inches is seen to be 180 000 pounds by following the ordinate for a diameter of 8 inches until it meets the radial line marked $1\frac{7}{8}$ inches, and reading off its value from the scale at the right.

The number of $\frac{7}{8}$-inch rivets in single shear is laid off at the top of the diagram, so that by following down any ordinate until the diagonal line (separating the two systems of horizontal and vertical ruling) is reached, the allowable shearing stress of the corresponding number of rivets may be read off by the scale on the right. Thus, the shearing value of 22 rivets is found to be 99 000 pounds. It may be added that the diagram as here printed is considerably reduced from the original size, on which more precise readings could be made. Usually, however, it is not necessary to read closer than 1000 pounds. On the left side the number of $\frac{7}{8}$-inch rivets in bearing is laid off to such a scale that by following any horizontal line until it intersects a line radiating from the upper left-hand corner on which the thickness of plates, or the linear bearing of the rivets, is marked, the equivalent number of rivets in shear may be read off on the scale at the top. For instance, the bearing stress of 12 rivets in a $\frac{1}{2}$-inch plate is very nearly equal to the stress of 14 rivets in single shear.

By combining the two preceding operations the value of the bearing stress of 12 rivets in a $\frac{1}{2}$-inch plate may be obtained by following down from the point of intersection to the diagonal line, and then reading the stress on the scale at the right, the

PIN-PLATE AND RIVET DIAGRAM.

Unit stresses:
Bearing of pin, 12 000 lbs. per square inch.
Bearing of rivets, 12 000 lbs. per square inch.
Shear of rivets, 7500 lbs. per square inch.

$\frac{7}{8}$-inch rivets:
Single shear, 4510 lbs.
$\frac{3}{8}$-inch bearing, 3940 lbs.
$\frac{7}{16}$-inch bearing, 4590 lbs.

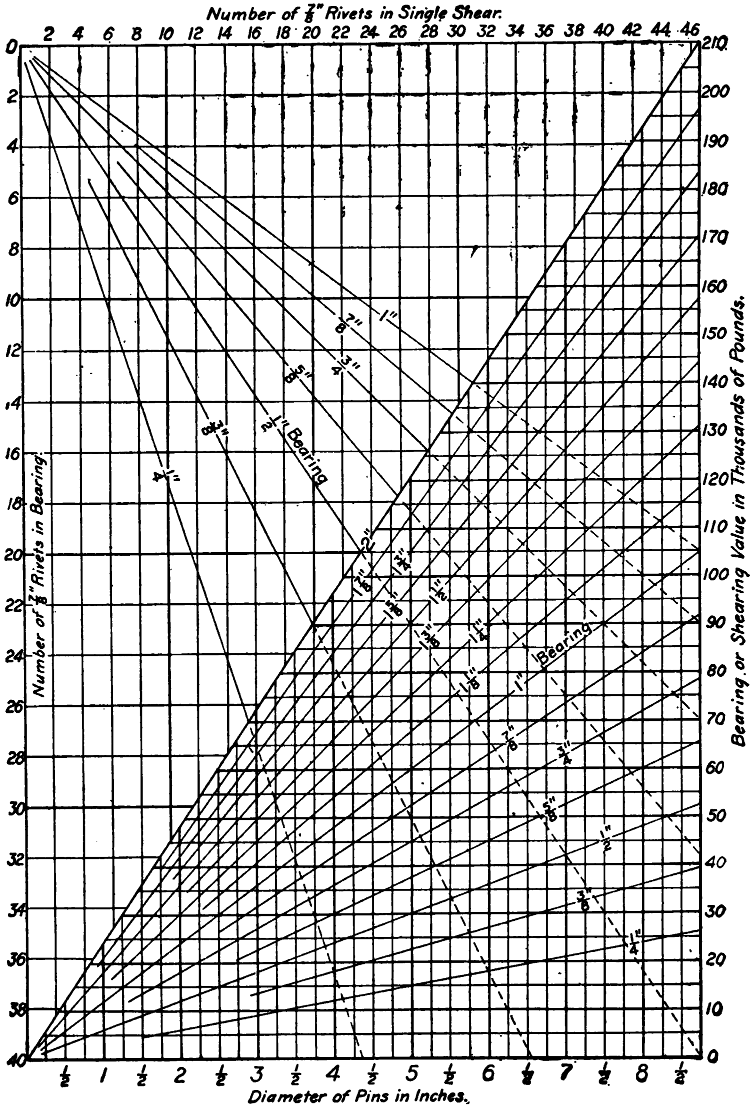


Fig. 29.

value being 63 000 pounds. Furthermore, all of the preceding operations may be combined. For example, the bearing value of a 6-inch pin with a bearing of $1\frac{1}{4}$ inches equals the shearing value of 20 rivets or the bearing value of 23 rivets in a $\frac{3}{8}$-inch plate. The upper radiating lines go beyond the diagonal in order to extend the limits of the diagram.

ART. 36. CONVENTIONAL SIGNS ON DRAWINGS.

Full lines show that they are visible, while invisible lines are represented by a series of dashes of equal length. In order to distinguish between invisible lines of the structure or object and the projecting lines it is desirable to use dashes about $\frac{1}{8}$ inch long for the former and about one-third as long for the latter. The appearance of a drawing is materially improved by making the spaces between the dashes uniform. In general these spaces should measure about $\frac{1}{32}$ inch or a little less than the smaller dashes. If the spaces are longer than the dashes, a line loses its apparent continuity if it is placed close to other lines of a similar character.

Feet are indicated by a prime ($'$), and inches by seconds ($''$), and these are usually placed on dimension lines having arrow points at the ends. These lines should be of two kinds: first, those marking the points, lines, or sections between which the measurement is to be recorded; and second, those drawn at right angles to the preceding lines, with an arrow at each end and the dimension marked at the middle. The former should have the same form as projecting lines, while the latter may either be the same or may be distinguished from both projecting lines and invisible lines of the structure by using very short dashes or elongated dots and spaces nearly or quite $\frac{1}{8}$ inch long. In constructing these lines the pen should be opened about twice as far as for the ordinary lines constituting the greater part of the drawing.

Center lines of plans, elevations or sections, or lines marking the position of sections whose forms are shown elsewhere, are appropriately indicated by the usual convention for traces of auxiliary planes, consisting of very long dashes, say $\frac{3}{8}$ inch, with two dots between them. Center lines of members or rivet lines are indicated either by very light full lines in black or by red lines of ordinary weight. The red lines on tracing cloth usually give faint lines on the blue print which may be readily seen. The sizes of dashes and spaces given above are those suitable for bridge drawings whose scale ranges from $\frac{1}{2}$ to 1 inch to the foot, and should be modified accordingly for scales beyond these limits.

When drawings are to be shaded by making some of the lines heavier than others, the following simple rule decides which lines are thus to be distinguished, plans being treated the same as if they were elevations: If a line separates two surfaces and there is an offset perpendicular to, and toward, the plane of projection in passing from the left-hand or upper surface to the right-hand or lower surface, the line (marking the offset) should be shaded. If the offset is in the opposite direction, that is, if the former surface is nearer the plane of projection (or farther from the observer) than the latter, the line is not to be shaded. When the offset is not perpendicular, as in the case of a beveled or chamfered edge, the form is usually indicated by the presence of diagonals or curves at the ends of the chamfer. If the line marks a rounded edge, its weight should be increased but slightly. Shading adds very materially to the realistic effect of a drawing and enables workmen not trained to the use of drawings to interpret them more readily. On account of the extra labor involved, shading is frequently omitted on shop drawings.

Clearness in detail drawings often demands that cylindrical, conical, spherical, or other curved surfaces should be covered

with shade lines spaced in accordance with the principles of shades and shadows in descriptive geometry.

Cross-sections are usually ruled with parallel lines, drawn light and full, as shown in Fig. 53, Art. 44. Sometimes a standard of section lining is adopted for different materials such as those shown in Fig. 12, Art. 17. The method frequently adopted where it is necessary to make the distinction is to mark those parts composed of any material other than that which constitutes the bulk of the structure by placing the name of the material either on or adjacent to them. When the section is so small that ruling will not appear to be suitable, the section is filled up solid. In order that adjacent sections so represented may appear distinct in form, a small space is left between them, although the shapes are really in perfect contact.

In order to give proper directions for the riveting, conventional signs are employed on the drawing. Two systems are in general use, one being known as OSBORN's code and published in OSBORN's Tables of Moments of Inertia as well as in nearly all of the handbooks, while the other is the Pencoyd system, which is given in the Pencoyd handbook and in the American Bridge Company's Standards for Structural Details.

Where the sign of a rivet head is surrounded by a broken circle of larger diameter, it represents the insertion of a washer to maintain uniform spacing between two angles acting together as a strut or tie. When the terms angles, channels, I-beams, etc., are not marked on the drawing, symbols are used having the form of the section. For additional information regarding shop drawings, see Art. 17 and Chap. X.

CHAPTER VI.

DETAILS OF PLATE-GIRDER BRIDGES.

ART. 37. GENERAL ARRANGEMENT.

The simplest form of a plate girder is composed of a vertical web plate to whose top and bottom are riveted horizontal pairs of angles, and to whose ends vertical angles are attached which serve to transmit the load to the supports. As the ratio of the depth of the web to its thickness increases, it becomes necessary to stiffen the web by fastening additional vertical angles at intervals along the span, these being also arranged in pairs on opposite sides of the plate. (See Fig. 30.) As the span increases, two or more web plates must be used and spliced end to end, while for long spans the flanges also require splicing.

For short spans one end of the girder is permitted to slide upon the support, a bearing or base plate being riveted to the bottom of the girder, and a bed plate bolted to the masonry or other support. When the span exceeds about 75 feet, it is desirable to make better provision for the expansion and contraction due to changes in temperature by introducing rollers between the bearing and bed plates, while for spans which are but slightly, if any, longer hinged bolsters are used in order to avoid the unequal pressure upon the rollers due to the deflection of the girder. At the same time the composition of the flanges and of other details is changed so as to provide for the increased stresses caused by greater loads and spans. (See Plate I, Art. 69.)

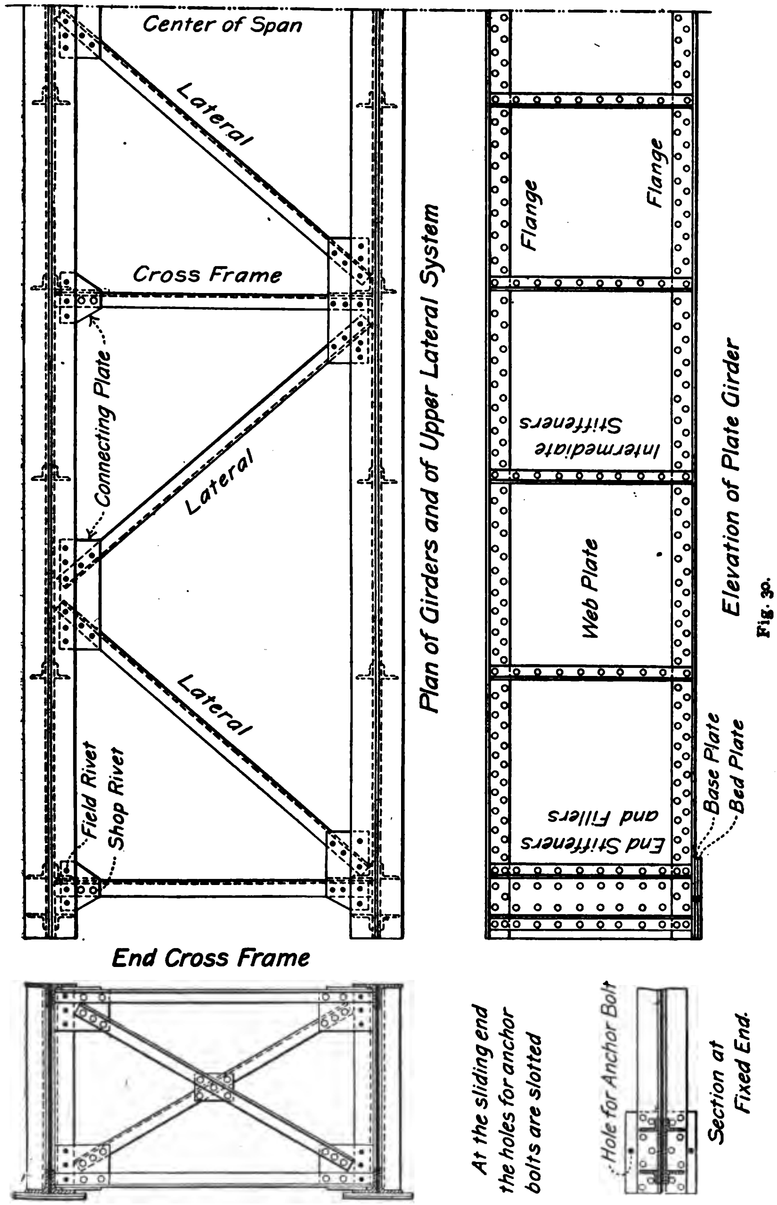

Center of Span
Lateral
Cross Frame
Connecting Plate
Lateral
Lateral
Field Rivet
Shop Rivet
Plan of Girders and of Upper Lateral System
Flange
Flange
Intermediate Stiffeners
Web Plate
End Stiffeners and Fillers
Base Plate
Bed Plate
Elevation of Plate Girder
End Cross Frame
At the sliding end the holes for anchor bolts are slotted
Hole for Anchor Bolt
Section at Fixed End.

Fig. 30.

A plate girder bridge consists of two or more girders connected together by one or two systems of lateral bracing, and by transverse bracing which comprises two or more cross-frames. In a deck bridge the railroad track or highway flooring rests directly upon the tops of the girders, while in a through bridge the floor is attached to the web plates. In one type of the through bridge, transverse girders, or floor beams are connected by gusset plates to the stiffener angles of the main girders to which, in turn, are attached the longitudinal beams or stringers that carry the cross-ties or the planking or floor plates. In another type of construction the transverse beams consist of rolled or built-up shapes that either form a solid or continuous floor, or they are placed so close together as to obviate the use of the stringers. As through plate-girder bridges can have only a lower lateral system, the upper flanges of the girders which are subject to compression must be held in line by means of bracing composed of the floor beams and their angle and gusset plate connections, which are extended up to those flanges for this purpose. When solid floors are employed, a similar arrangement is necessary at corresponding intervals.

ART. 38. THICKNESS OF WEB PLATES.

Experience shows the importance of specifying that the thickness of web plates in railroad bridges shall not be less than three-eighths of an inch, while those in highway bridges and buildings shall not be less than five-sixteenths of an inch. It would be better if three-eighths of an inch were also made the minimum thickness for important highway bridges. In girders carrying heavy loads the magnitude of the vertical shear will frequently require a greater thickness than the minimum value.

Usually the thickness of the web is made the same throughout, as it simplifies the shopwork, the excess of material being offset by the saving in labor. In special cases, however, it may

be necessary to use such a thick web at the ends that it will be economical to vary the thickness either once or twice in the half span. When this is done, filler plates must be placed between the web and the angles on one or on both sides of the flanges, so as to maintain a constant distance between the backs of the flange angles. An illustration of this may be found in the Engineering Record, vol. 43, page 102, Feb. 2, 1901. The web thicknesses of the middle girder are $1''$, $\frac{5}{8}''$, and $\frac{3}{8}''$, respectively, and two fillers $\frac{3}{16}''$ thick are used under the flange angles along the $\frac{5}{8}''$ web, while those along the $\frac{3}{8}''$ web are $\frac{5}{16}''$ thick. The outer girders of the same bridge have web plates $\frac{1}{2}''$ and $\frac{3}{8}''$ in thickness, and one filler plate $\frac{1}{8}''$ thick is used under the outer flange angles only.

In the design of plate girders the use of somewhat thicker web plates than is customary should be encouraged because inspectors with extended experience report that they find that the web always gives out first, even in cases where the flanges are thinner. This may be done by specifying that a part of the web section shall be taken as flange area in accordance with the theory of flexure. A thin web gets out of shape more readily in handling, deteriorates more rapidly after it begins to rust, and is more easily injured in case of accident.

Sometimes, instead of changing its thickness, the web plate is reinforced near the ends by riveting a plate on each side of it between the upper and lower flange angles.

ART. 39. COMPOSITION OF FLANGES.

The simplest flange of a plate girder is shown in Fig. 31 and consists of a pair of angles riveted to the web plate with their backs projecting slightly beyond it, so that if the edge of the plate is not perfectly straight, there may be no interference with anything resting upon or attached to the flange.

When additional flange area is required, one or more cover plates are usually riveted to the horizontal legs of the angles as shown in Fig. 32. The number of cover plates should generally be limited so as not to require rivets longer than five times their diameter in order that they may entirely fill the holes when driven. As the stress in the covers must be transmitted indirectly from the web through the angles, it is desirable to use angles whose sectional area shall be equal to or greater than that of the cover plates, and if this is not possible, the largest angles obtainable should be used. Since the bending moment in the girder requires cover plates of different lengths which in turn necessitates notching the cross-ties unequally for deck girders of railroad bridges, the flange has occasionally been

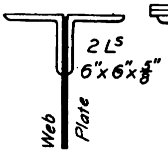


Fig. 31.

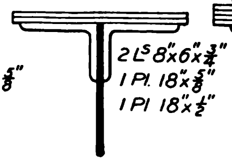


Fig. 32.

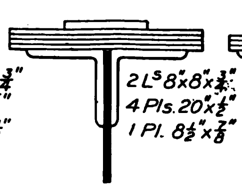


Fig. 33.

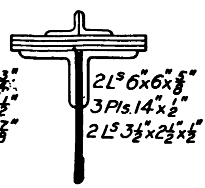


Fig. 34.

modified by making the outer plate narrower than the rest and extending it from end to end of the girder with filler plates placed under it beyond the shorter cover plates. See Fig. 33 and Engineering Record, vol. 44, page 6, July 6, 1901. In Fig. 34 the same object is secured by means of two small angles, the ties being notched over the vertical legs of the angles. The section of the flange at the end of the girder is shown in Fig. 38. With both of these forms of flange the load is brought nearer to the center of the flange. The cross-ties are all alike, and this is a convenience in the removal of ties or in bunching them to secure access below the floor for repairs. The rivets are countersunk so as to avoid interference with the bearing of the cross-ties.

Fig. 35 gives a section used by the Chicago, Milwaukee, and St. Paul Railway in the plate-girder bridges carrying its elevated tracks in Chicago. The top pair of angles extends from end to end, while the lowest pair is the shortest. This flange also secures an even bearing for the track ties without the necessity of boring holes for rivet heads, as is the case generally when cover plates are used, the cross-ties being held in place laterally by notching over the projection of the web plate which extends $\frac{3}{4}''$ or $\frac{7}{8}''$ above the flange angles. The bottom flange has 2 angles $6'' \times 6'' \times \frac{5}{8}''$, and 2 cover plates $14'' \times \frac{5}{8}''$. In another girder of larger span the top angles of the upper flange are increased to $8'' \times 8'' \times \frac{5}{8}''$, thereby securing three rows of rivets to connect the flange angles to the web.

Another method of reducing somewhat the total thickness of cover plates is to insert a vertical plate between each flange angle and the web as illustrated in Fig. 36. This arrangement

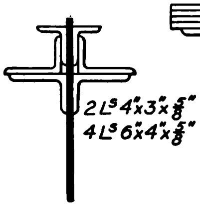


Fig. 35.

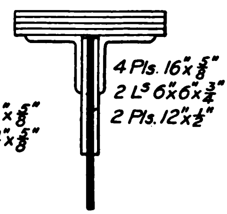


Fig. 36.

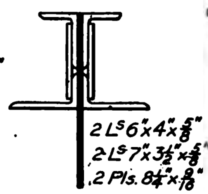


Fig. 37.

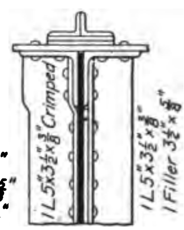


Fig. 38.

has the advantage of permitting three or four rows of rivets to transmit the increments of flange stress from the web to the flanges. The vertical plates are usually made about twice as wide as the angles. Probably the largest flanges of this form which have been constructed are those in the Maiden Lane bridge at Albany, described in Engineering Record, vol. 40, page 474, Oct. 21, 1899. The angles are $8'' \times 8'' \times 1''$, and the vertical plates $16'' \times \frac{5}{8}''$, while the cover plates are $27''$ wide, three being $1''$ thick and the other $\frac{13}{16}''$ thick.

The type of flange which has been used in the heaviest plate girder probably ever built, and in some other girders of very long span, is illustrated in Fig. 41, Art. 41. It consists of 4 angles, with one or more pairs of vertical plates on the faces of the angles, together with a number of cover plates. In the 103-ton plate girder, which is the middle one in a four-track through bridge on the New York Central and Hudson River Railroad over the Clyde River east of Lyons, N. Y., each flange consists of 2 angles $8'' \times 8'' \times 1''$, 2 angles $6'' \times 6'' \times 1''$, 6 side plates $12\frac{1}{2}'' \times \frac{1}{2}''$, only two of which extend the full length, and 10 cover plates, one of which extends the full length of the girder, 7 plates being $24'' \times \frac{5}{8}''$, and 3 plates $24'' \times \frac{1}{2}''$. The flanges of the outside girders have 2 angles $8'' \times 8'' \times \frac{3}{4}''$, 2 angles $6'' \times 6'' \times \frac{3}{4}''$, 2 side plates $13'' \times \frac{1}{2}''$, full length, and 5 cover plates, two being $24'' \times \frac{5}{8}''$, and 3 plates $24'' \times \frac{1}{2}''$. This bridge is described and illustrated in the Engineering Record, vol. 43, page 102, Feb. 2, 1901.

Another girder on the Erie Railroad, whose span is $125'\ 2\frac{1}{2}''$, has an upper flange of 4 angles $8'' \times 6'' \times \frac{5}{8}''$, and 4 cover plates $18'' \times \frac{9}{16}''$, no vertical plates being used. The long legs of the angles are horizontal. See Fig. 41, or Engineering Record, vol. 41, page 565, June 16, 1900.

A modification of this type in which the side plates are included, but the cover plates omitted, is given in SCHAUB'S specifications. The web plate is to project $\frac{1}{2}$ inch above the angles to engage the notches in the railroad track ties. An interesting example of upper flanges without cover plates is given in the Boone Viaduct. See Fig. 37, and Engineering News, vol. 46, page 117, Aug. 22, 1901.

The flanges shown in Figs. 37 and 41 have another advantage in permitting the lateral system to be attached to one of the lower angles, and thus avoiding the trouble from loose rivets in the bracing caused by the deflection of the cross-ties.

Dapping or notching the cross-ties over the cover plates or angles weakens them when the girders are spaced farther apart than the track rails. In those flanges where the cross-tie is notched over the projecting web, or the short vertical legs of bearing angles, as in Fig. 34, the cross-tie takes its bearing outside of the narrow notch, thus preserving its full strength. Less labor is also required in this case, as it is not necessary to bring the top of the notch to an even surface.

ART. 40. WEB STIFFENERS.

As pointed out in Art. 57, the theory of the distribution of stresses in the webs of plate girders and of the functions of intermediate web stiffeners is in an unsatisfactory state, and as a natural result the practice in the use of stiffeners varies considerably. An examination of the drawings of a large number of plate-girder railroad bridges of recent design indicates that for intermediate stiffeners, $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ angles are mostly used for spans below 50 feet, $5'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ angles for spans from 50 to 100 feet, and $6'' \times 4'' \times \frac{3}{8}''$ angles for spans above 100 feet. Thicknesses of $\frac{7}{16}''$ or $\frac{1}{2}''$ are employed in very few cases. In general the stiffeners for highway girders and buildings are somewhat lighter.

The end stiffeners usually consist of four angles for ordinary spans, one pair being placed at each end of the shoe or bearing plate, while for long spans two additional pairs of angles are placed midway between them, as shown in Figs. 43, 46, 48, and 49, Arts. 43 and 44.

Fillers are always employed under the end angles, so that the latter may be riveted on straight, as shown on the right of Fig. 38. Intermediate stiffeners are sometimes crimped over the flange angles, as indicated on the left side of the same figure, while in other cases fillers are used. When stiffeners connect

to cross-frames, it is preferable to use fillers. Some engineers prefer to limit crimping to about $\frac{1}{2}$ inch, using fillers only for the excess thickness of the flange angles. When vertical side plates are used in the flanges, fillers of the same thickness are inserted between the upper and lower side plates, while the stiffeners are crimped over the flange angles.

For a novel arrangement of end stiffeners and fillers in which the angles form a continuation of the upper flange angles without reduction of thickness, see Railroad Gazette, vol. 28, page 769, Nov. 6, 1896, and the inset of Engineering News for Aug. 20, 1896. In the former case the intermediate stiffeners, which are about 7 feet apart, do not extend over the upper flange angles.

ART. 41. WEB SPLICES.

The processes of manufacture and the available equipment necessarily impose limitations to the size and weight of web plates, so that large plate girders require a number of web splices. The simplest form of splice shown in Figs. 45 and

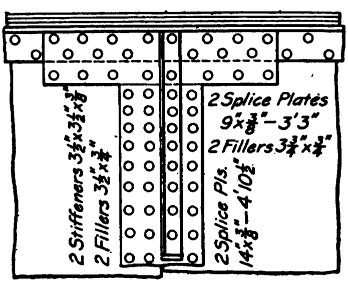


Fig. 39.

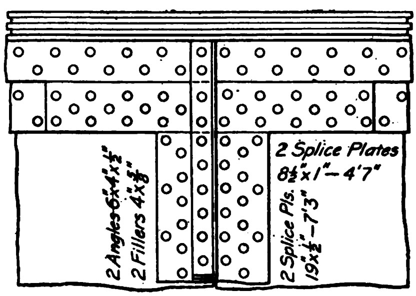


Fig. 40.

49, Art. 44, consists of plates whose length equals the clear distance between the flange angles, and which are riveted to each of the two web plates by two or more rows of rivets. A pair of stiffener angles is generally attached to the splice

A more efficient splice is that designed in Art. 56 (see Fig. 68). The two flats riveted to the vertical legs of the flange angles not only splice the part of the web not reached directly by the other plates, but add considerably to the strength of the whole splice, since the value of any rivet in resisting the bending moment at the joint is proportional to the square of its distance from the neutral surface. A rivet at the neutral surface can resist shear, but no bending moment. Such a splice is used on the girders of the New Kinzua Viaduct described in Engineering Record, vol. 42, page 510, Dec. 1, 1900. If additional strength be required, the flats over the flange angles

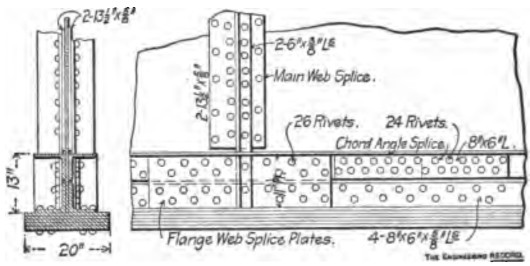


Fig. 41.

may be widened so as to engage one or two extra rows of rivets, filler plates of the same thickness as the flange angles being placed between them and the web. See Fig. 39 and also Engineering News, vol. 30, page 140, Aug. 17, 1893. Fig. 40 shows the web splice in a very heavy girder whose flanges contain side plates.

Longitudinal splice plates for the web are well adapted to the case where the flange has four angles, as in Fig. 37, Art. 39. The splice illustrated in Fig. 41 is the one used on the plate girder of $125' \, 2\frac{1}{2}''$ span referred to in Art. 39.

On the Duquesne approach of the Monongahela River bridge at Rankin, Pa., the splices of some girders, whose span is about

118 feet, were made by placing the longitudinal plates alongside of the vertical flange plates, the vertical splice plates being put between these. The use of vertical flange plates requires either the arrangement just mentioned, thereby reducing the relative effectiveness of the splices, or the use of many fillers to move the longitudinal splice plates farther from the neutral surface of the girder. The total thickness of fillers may be reduced by crimping the stiffeners over the longitudinal splice plates.

It is customary to place a pair of stiffeners at each web splice, but in the girders last mentioned most of the splices are located in the spaces between the stiffeners.

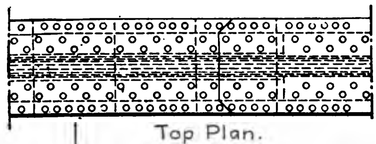
ART. 42. FLANGE SPLICES.

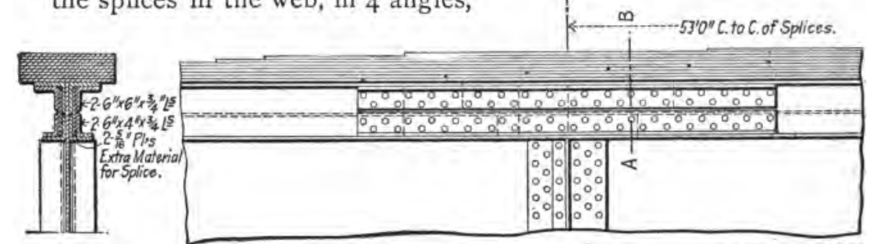
For spans of plate girders less than 60 feet, it is possible to avoid splices in the flanges, as angles and cover plates extending the full length required may be readily obtained. It is frequently economical to pay the additional price that may be asked for extra lengths of such material in order to avoid splices altogether, or to reduce their number to a minimum. No two pieces of either the web or the flange should be spliced within a certain distance of each other, that distance being such as to enable one cut to be fully spliced before the next one is reached.

When each flange consists of only two angles and cover plates, it is customary to splice the outside angle on the left of some web splice, and the inside angle on the right of the web splice in one of the flanges, and to reverse this arrangement in the other flange. The flange angles are usually spliced by means of cover angles whose roots are rounded to fit the fillets of the other angles. The most convenient arrangement for the cover splices is to place them so that in each case the

outer cover near the splice may be extended a sufficient distance to form the splice plate.

Fig. 42 shows the splice of a flange with an exceedingly heavy section. On account of the necessity of shipping this girder in three pieces, the entire flange splice had to be confined to a comparatively short length. The figure shows the location of the splices in the web, in 4 angles,





Section A-B. Top Chord Splice. Center Girder. THE ENGINEERING RECORD.

Fig. 42.

2 side plates, and 6 cover plates, besides the ends of the remaining 4 cover plates, some of which are extended so as to act as splice plates. The composition of this flange of a 103-ton girder was given in Art. 39.

In Fig. 72, Art. 61, and on Plate I, details are shown in which both a cover angle and a flat placed on the opposite side are used to splice one of the flange angles.

ART. 43. LATERAL AND TRANSVERSE BRACING.

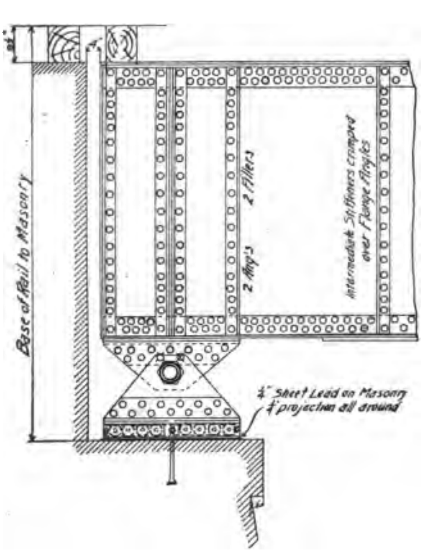
The upper and lower flanges of deck plate girders are held in line by means of a series of braces, each one being composed of one or two angles, which together with the flanges form horizontal trusses, known as the upper and lower lateral systems respectively. These systems are most frequently of the Warren type, the panel points of the upper system being

directly above points which are midway between the panel points of the lower system. Transverse or sway bracing is placed at the ends of the girders and at intervals between. These cross-frames usually consist of two horizontal struts and two diagonals, which together with a stiffener on each girder form a rigid rectangular panel. (See Fig. 44.) Similar horizontal struts are often inserted at other points between the cross-frames. The end cross-frames sometimes have diagonals composed of channels instead of angles, while in exceptional cases a solid web plate has been employed. The lower horizontal strut is occasionally omitted in intermediate frames.

In double-track deck bridges the inner girders supporting each track are connected by struts of single or double angles to keep them at a fixed distance apart, no diagonals being employed. These are also shown in Fig. 44. The object of the $\frac{3}{4}''$ fillers inserted between the lateral connecting plates and the flanges is to secure more clearance between the lateral braces and the cross-ties.

Fig. 45 gives a view of two adjacent deck plate-girder bridges on the Baltimore and Ohio Railroad, taken before the track was put in place on one of them. This shows the general character of the upper lateral system and the cross-frames as well as the form of their connections to the girders.

In double-track through bridges both tracks are generally supported between two girders, and the lateral system is then preferably made of the rectangular type, two sets of diagonals being inserted between each pair of floor beams, the latter acting as the struts of the system. The same arrangement is used for single-track through girder bridges, the laterals being riveted to the stringers in both of these cases so as to transfer directly to the girders any stresses due to the braking of trains, or to other longitudinal forces, which might otherwise cause the floor beams to bend horizontally.



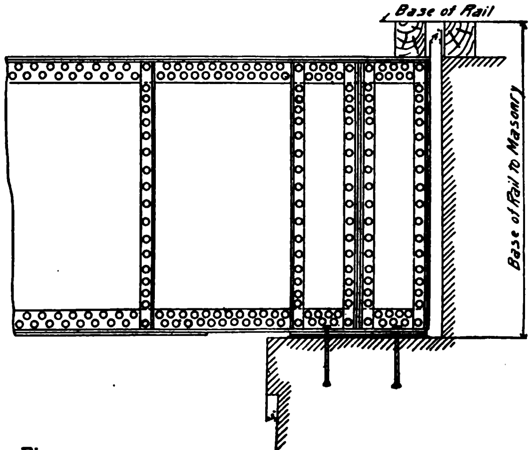
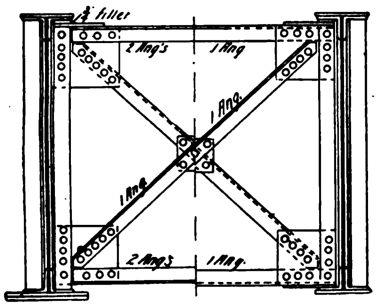
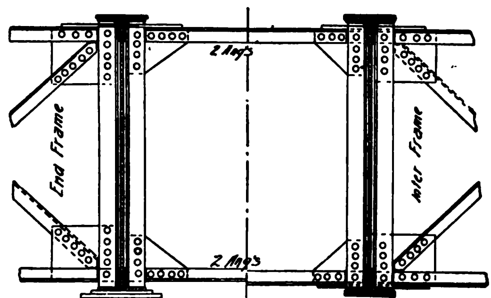


Fig. 43.



End Frame Intermediate Frame



Section showing connection between Girders for 2 or more tracks.

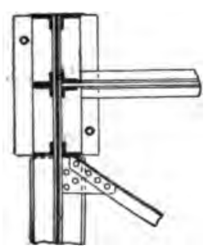
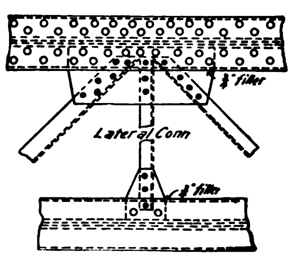


Fig. 44.



From Standards of New York Central and Hudson River Railroad.

The gusset plates, used as transverse bracing for the upper flanges of through plate girders will be described in Art. 45,

Fig. 45.

since in the best construction these gusset plates form an integral part of the floor system.

ART. 44. EXPANSION BEARINGS.

For short spans a base plate is riveted under the bottom flange at each end of the girder; this rests upon a bed plate that distributes the pressure to the masonry. At one end the anchor bolts hold the base plate rigidly in position, while at the other end the holes are slotted so as to permit the base plate to slide longitudinally on the bed plate under the influence of temperature changes and deflection.

When the span exceeds about 75 feet, friction rollers are introduced, and in order to insure a uniform distribution of pressure, the best practice requires at the same time the use of a pin bearing, the rollers being placed between the pedestal and the bed plate. One standard form of this bearing is shown in Fig. 46. In both shoe and pedestal, three webs composed of

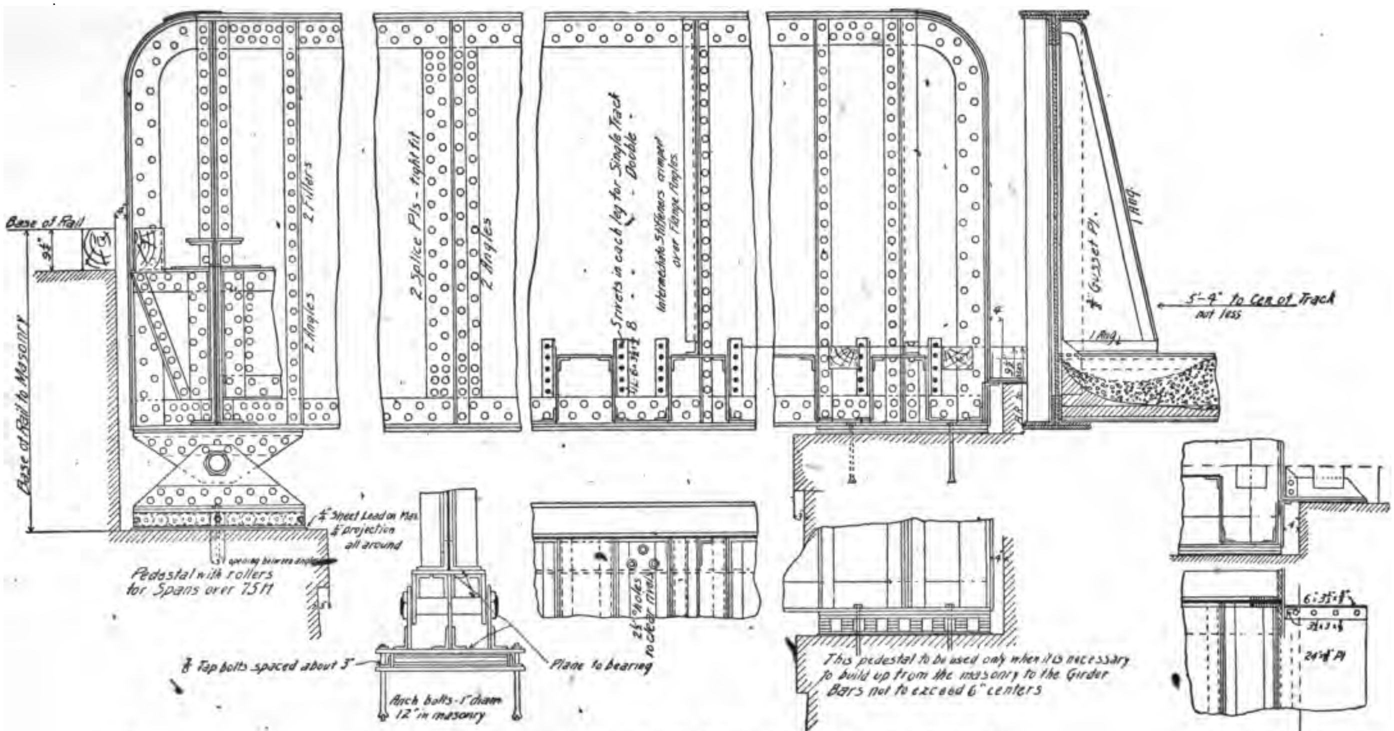


Fig. 46. From Standards of New York Central and Hudson River Railroad.

vertical plates are connected to the horizontal bearing plate by means of angles. The rollers are kept at the proper clearance by two side bars which engage tap bolts in the ends of the rollers. Three tie rods hold the side bars from separating. Angle irons are placed around the nest of rollers to form a dust guard, those on the sides also acting as guides to prevent lateral motion of the girder. WADDELL specifies that the rollers are to be inclosed in dust-proof boxes filled with oil of a given quality.

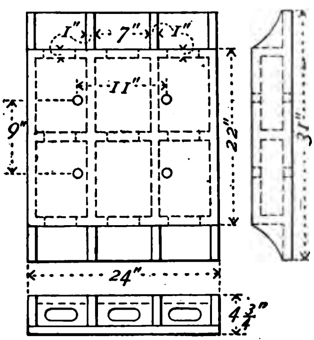
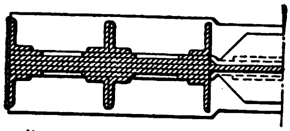


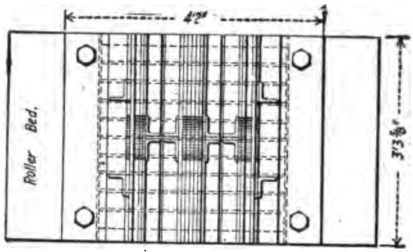
Fig. 47.

At the fixed end a cast-iron base is used whose height equals that of the rollers and bed combined so as to make the height of masonry the same at both ends of the span. One form of such a base is given in Fig. 47.

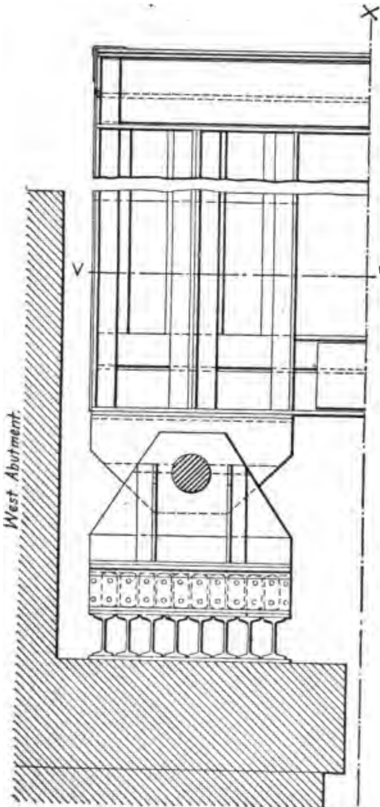
The complete detail drawings of a similar bearing designed for a span of 100 feet are given on the insets of Engineering News for July 8 and 15, 1897. The webs are stiffened by inclined diaphragms on both sides, the rollers are relatively larger, and the dust guard is differently arranged, the side bars being replaced by angles. The details of another bearing are shown in Railroad Gazette, vol. 27, page 771, Nov. 22, 1895. In this case there are only two webs, each one connected to the bearing plate with two angles. The webs are joined by a central vertical diaphragm which is arranged to take continuous bearing on the pin. The rollers move on flats riveted to the bed plate, the grooves between which gather the dust that may pass the guards and facilitate its subsequent removal. The rollers are grooved in the middle and engage a flat riveted to the base plate of the pedestal as well as the middle flat on the bed plate which is higher than



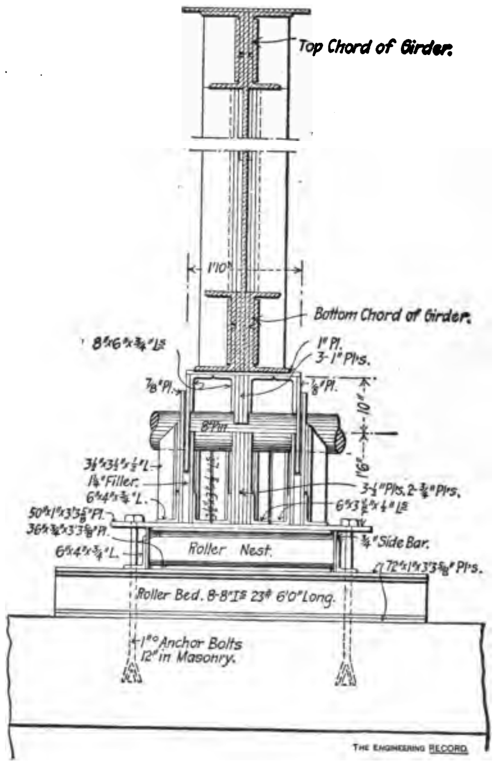
Horizontal Section on Line V-V.



Plan of Pedestal.



Elevation.



Sectional Elevation on Line X-X.

Fig. 48.

the rest, thus preventing lateral motion of the girder and relieving the side angles of the dust guard from that duty.

Fig. 48 illustrates the bearing of a 103-ton plate girder on the New York Central and Hudson River Railroad. The pedestal has five webs, a transverse diaphragm between the inner webs, the outer webs being stiffened by vertical angles on the outside. The rollers are segmental with parallel sides, and are 6″ in diameter and 3″ thick. The roller bed contains closely spaced I-beams in order to distribute the pressure on the masonry over a larger area. At the middle pier of this two-span bridge the adjacent shoes of the fixed ends of the girders engage the pins of a single pedestal whose length is not quite double that of those on the abutments. In order to secure increased web bearing on the base plate of the shoe the lower part of the web is reinforced by two plates, and the lower flange angles are crimped around them as indicated in the horizontal section.

Fig. 50 shows beds containing piles of 4″ × 1″ flats spaced 4″ apart in the clear, offset at the ends so as to secure larger bearing areas for the bed plates. The construction of the roller dust guard is shown in Fig. 49, both bearing plates above and below the rollers having $\frac{5}{16}$″ shoulders, while the side bars extend $\frac{3}{16}$″ beyond the rollers.

Fig. 51 contains the end elevation, transverse section, and side elevation of a cast-steel expansion bearing which was designed for a deck girder with a span of 112 feet. The pin has a diameter of 6 inches, and is held in place by a ring one inch thick made in two sections. The thickness of metal in the cast pedestal is 2 inches. Another bearing of the same kind is shown in Fig. 52, and was designed for a single-track through girder whose span is 125′ $2\frac{1}{2}$″. The details of the steel castings indicate that the 3″ pin has continuous bearing. The pin is

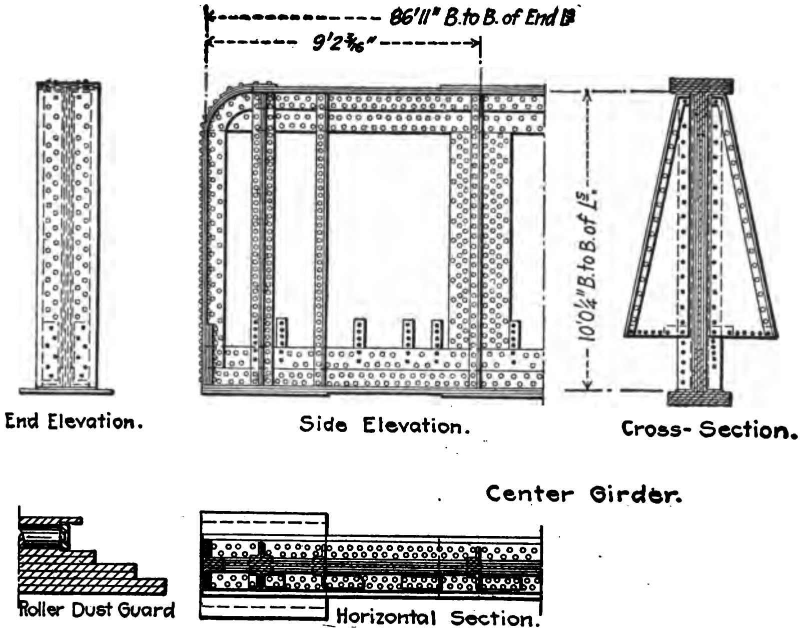

86'11" B. to B. of End Ls
9'2 3/16"
10'0¼" B. to B. of Ls.
End Elevation.
Side Elevation.
Cross-Section.
Center Girder.
Roller Dust Guard
Horizontal Section.

Fig. 49.

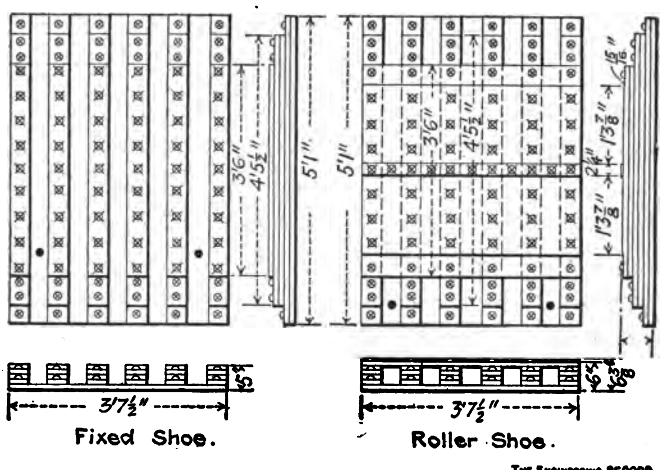

3'6"
4'5½"
5'1"
5'1"
3'6"
4'5½"
15/16"
1'3 7/8"
2¼"
1'3 7/8"
3'7½"
3'7½"
Fixed Shoe.
Roller Shoe.
THE ENGINEERING RECORD

Fig. 50.

omitted in the outline sketch of the assembled bearing. In Engineering Record, vol. 40, page 414, Sept. 30, 1899, may be found an illustration of a cast-steel bearing in which the pin is flattened on the lower side, while in Railroad Gazette, vol. 25, page 684, Sept. 15, 1893, another one is given in which the pedestal has a semi-cylindrical projection that enters the concave bearing of the shoe and thus replaces the pin.

A novel design was made in 1900 by the American Bridge Company, under the direction of the bridge department of the Lehigh Valley Railroad, for the support of a 116-foot deck plate-

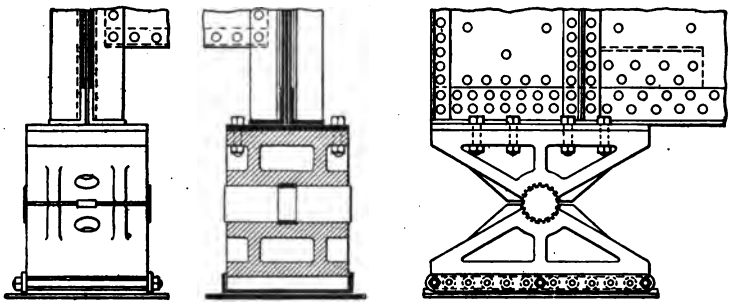


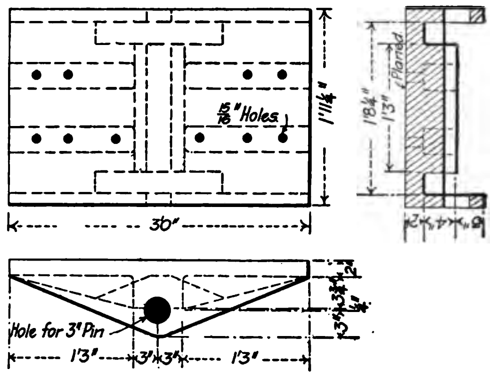
Fig. 51.

girder bridge at Mauch Chunk, Pa. As shown in Fig. 53 the end shear is transferred by pin plates directly from the web and end stiffeners to the 9-inch pin of the shoe or pedestal. By this arrangement the distance from the bottom flange of the girder to the masonry is reduced to 9 inches. The distance from the base of rail to the masonry is 10 feet $7\frac{1}{4}$ inches.

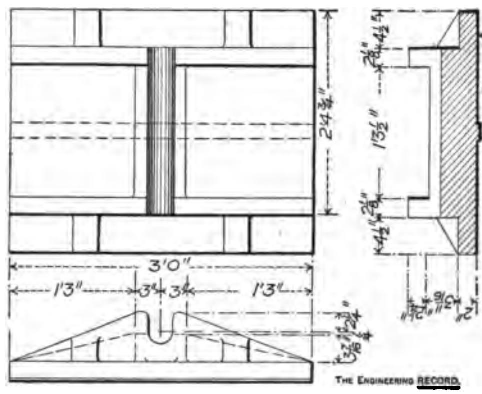
Another design embodying some unusual features is that of an expansion bearing for a plate-girder bridge of $114' 6''$ span on the Chicago, Milwaukee and St. Paul Railway at Janesville, Wis. A planed phosphor-bronze plate 8 inches square and $1\frac{1}{2}$ inches thick is inserted between the upper and lower castings, permitting the upper casting to move longitudinally and the girder to deflect without disturbing the pedestal bearings. The

castings have projecting longitudinal flanges which inclose the bronze plate and prevent lateral displacement. The plate is tap bolted to the lower casting. It will be observed that the object of these bearing plates is to replace both the pin and the friction rollers. See illustration in Engineering Record, vol. 44, page 6, July 6, 1901.

Fig. 54 shows a standard hinge joint of cast steel for spans from 50 to 65 feet, introduced on the Northern Pacific Railway. For spans over 65 feet segmental rollers 12 inches in diameter are used, the form of bearing being the same as that used for riveted trusses. See Plate II, Art. 69. These spans are considera-



Top Casting.



Bottom Casting.

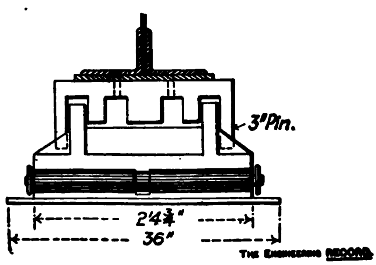


Fig. 52.

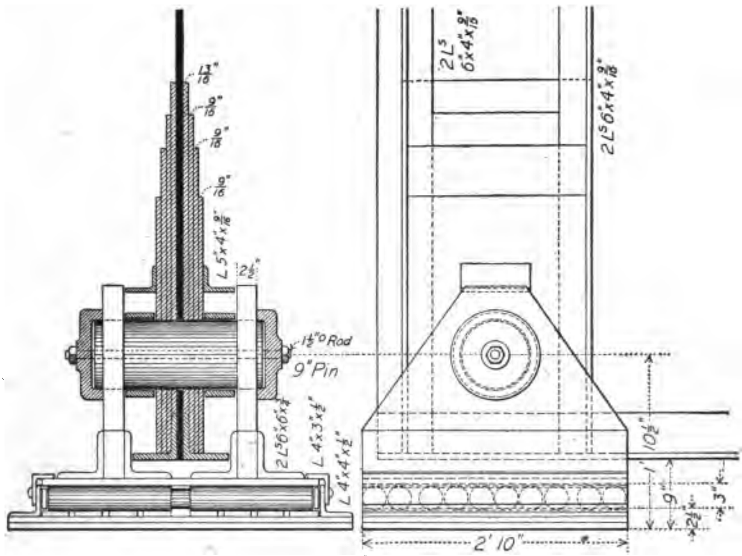
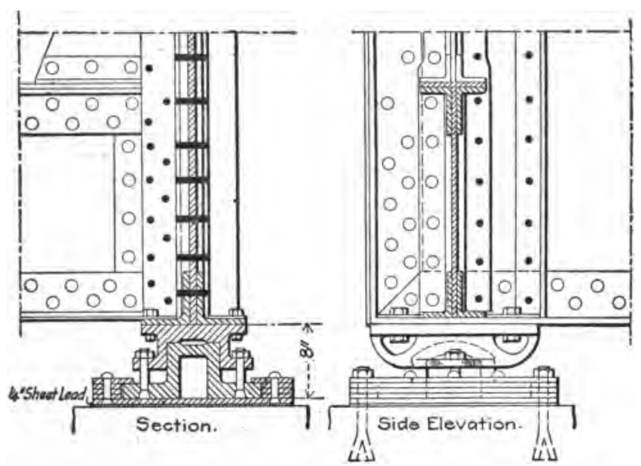


Fig. 53.



THE ENGINEERING RECORD.

Fig. 54.

bly less than the usual limit assigned in practice to the use of expansion bearings. The details of a rocker bearing, formerly a standard on the same railroad, may be seen in Engineering Record, vol. 37, page 5, Dec. 4, 1897. The rocker consists practically of a double pin 4″ in diameter and 8″ high.

ART. 45. FLOOR SYSTEM.

The floor system of through railroad plate-girder bridges in most extensive use consists of floor beams and stringers, the

Fig. 55.

latter supporting the cross-ties on their upper flanges. The floor beams are generally spaced from 12 to 18 feet apart. The spacing is slightly less in single-track than in double-track bridges. The general arrangement of the floor system and its connection with the girders is clearly shown in Fig. 55, the view being taken before one of the tracks was put in place. The bridge is on the Baltimore and Ohio Railroad.

A floor beam is in reality a plate girder of short span, as indicated in Fig. 56. This illustration is a part of the standard details adopted by the New York Central and Hudson River Railroad. The web is spliced near each end in order to make an efficient connection with the main girders, both to transfer

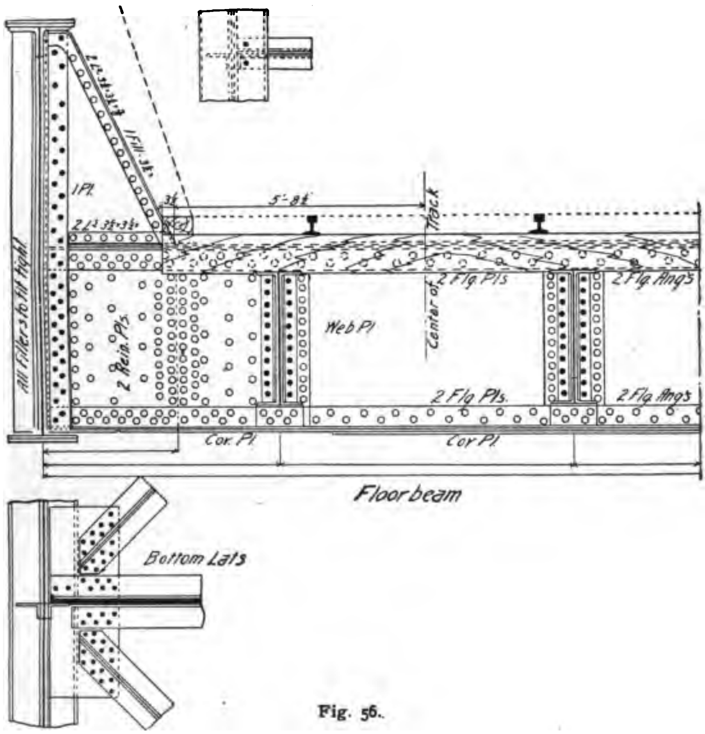


Fig. 56.

the shears and to stiffen the upper flange of the girders. As shown in the figure the triangular gusset plate is an upward extension of the end web plate, while the web splice plates are extended from the girder to the inside of the nearer stringer, to reinforce the web plate. This reinforcement is not needed in a single-track bridge. The outer edge of the gusset plate is stiffened by a pair of angles whose upper ends are bent over and

riveted to both the stiffeners and flange angles of the girder. This is more effective than the common arrangement in which these angles are cut off where they meet the edge of the stiffeners. Sometimes the gusset plate is made too narrow and not even extended up to the flange angles as it should be, while in other cases this plate is separate from the web plate of the floor beam, and only connected to it indirectly by short horizontal angles riveted to the flanges. Such a connection develops tension in some of the rivets, which should be avoided when possible.

When the lower flange of the girder contains four angles, and it is desirable to keep the bottom of the floor beam as low as

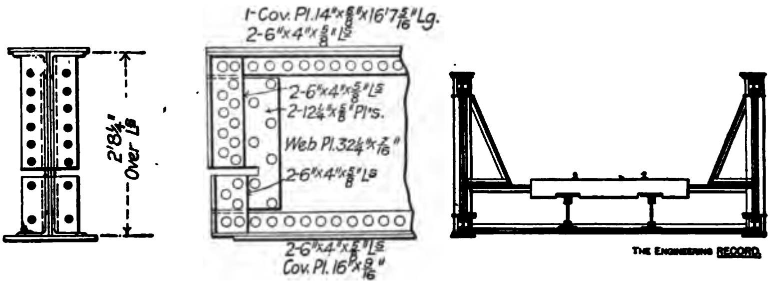


Fig. 57.

possible, the end of the floor beam must either be slotted, as in Fig. 57, or the connection to the girder may be omitted below the top angles of its lower flange, as in Fig. 58, since some of the shear may be transferred to the stiffeners above the floor beam, on account of the continuity of the end web plate.

The stringers may consist either of I-beams or be built up like a plate girder, in which case it is preferable to use no cover plates, and to allow the web plates to project above the flange angles. The left end of Fig. 46, Art. 44, shows a bracket, in line with the stringer and beyond the end floor beam, which carries the end track tie on the bridge.

In another arrangement of the floor system of through plate-girder bridges the ties rest on horizontal shelf angles riveted to the web near the lower flanges, and the gusset plate stays are attached to the transverse struts of the lateral system, which are made as deep as the track will allow, in order to give the needed stiffness. The method is objectionable on account of the warping of the timber, in spite of all precautionary appliances to prevent it, while at the same time the cross-ties are liable to be pushed over in case of derailment, on account of their greater depth required by the increased span of the ties.

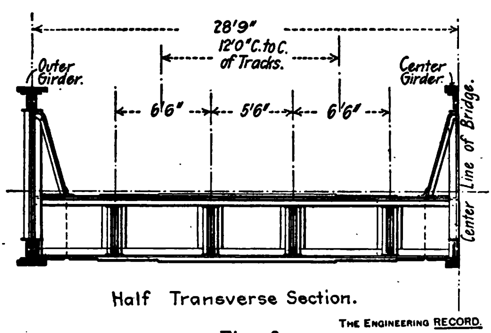


Fig. 58.

It is worse still to rest the cross-ties on the bottom flanges, since the constant springing of the floor tends to weaken the flanges and to loosen the lateral bracing.

In deck bridges the cross-ties rest directly upon the top flanges of the girders, two girders supporting each track. (See Fig. 43 in Art. 43.) This is the plan almost universally adopted. On the Boston and Maine Railroad, however, the standard deck-girder bridge contains floor beams and stringers. The floor beams are riveted to the webs of the girders at such a height that the stringers, which rest on top of them, are about even with the tops of the girders. The cross-ties extend over both stringers and girders, the latter, spaced 9 feet apart, acting thus also as safety stringers. There is but one lateral system, it being in the plane of the bottom flanges of the floor beams. The upper flanges of the girders are stayed by transverse web connections to the stringers directly over the floor beams.

A similar arrangement is adopted in the long span deck-girder bridges of the approaches to the Monongahela river bridge at Rankin, Pa. The track is on a curve, requiring the girders to be spaced 17′ 3″ or more apart. The stringers are 6′ 6″ apart and follow the track as nearly as possible, the track running close to the inner girder at its ends and the outer girder at its center.

In highway bridges floor beams are generally used, and to these are attached steel I-beams or wooden joists spaced relatively close together to give adequate support to the wooden floor planks or to the buckle plates which carry some form of paved floor. The sidewalks outside of the girders are carried on brackets or cantilever extensions of the floor beams. In the Journal of the Association of Engineering Societies, vol. 21, page 62, Aug., 1898, an illustration is shown of some grade-crossing work, in which the floor beams are dropped down below the lower flanges of the girders about half their depth in order to save head room over the railroad tracks below. The floor beams are placed midway between the clearances required by the railroad.

A large number of standard railroad bridge floors for deck and through bridges are described and illustrated by 14 plates, showing partly dimensioned details, in the report of a Committee of the Association of Railway Superintendents of Bridges and Buildings, published in BERG's American Railway Bridges and Buildings, pages 645–669, reprinted from the Proceedings of the Association.

ART. 46. SOLID FLOORS.

Solid floors in railroad bridges include many different types of continuous metal floors which support the rails on ordinary cross-ties in ballast. In some cases the ballast is omitted, and the cross-ties rest on the metal floor, while in other cases the

cross-ties are also omitted, and the rails are bolted directly to the metal floor.

In the earliest type used in this country old track rails were laid close together on top of the girders of short span bridges, and on which the ballast was spread, thus securing a continuous track free from the objections inhering in the transition to and from the standard wooden bridge floor supported on stringers. Floors of this kind were built as early as 1874.

The next form, which was introduced in 1887, consists of metal troughs built by riveting trough plates that are alternately inverted, as indicated in Fig. 59. The cross-ties are sometimes laid directly in the troughs, but more frequently bedded in ballast. In the following year the New York Central and Hudson River Railroad began building solid floors with continuous

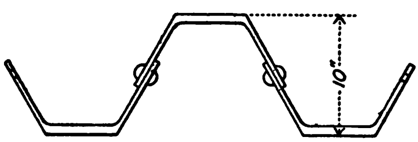


Fig. 59.

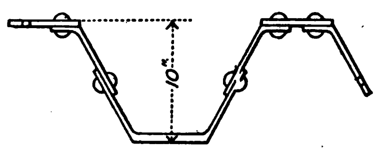


Fig. 60.

ballast, adopting the above section for deck bridges and the rectangular trough section, consisting of plates and angles (Fig. 64), for through bridges. The latter section is better adapted to being hung between the girders by connecting plates and angles than any form having inclined sides, and its depth may be readily increased to give the required strength for any given load and span. (See Fig. 46 in Art. 44.) This railroad was the first to adopt solid floors as a standard, and has continued to the present time to use the rectangular troughs with ballasted track where the depth of flocr is limited.

Fig. 60 gives a trough section composed of a flat, an obtuse angle, and a trough plate, which was designed to be easily manufactured and power riveted, adjustable for depth and spacing,

absolutely water tight, and to avoid rivets in the tension side with the consequent reduction in strength. The sections in Figs. 61 and 62 have a fixed depth, but are readily adjustable horizontally by varying the widths of the flats, the latter re-

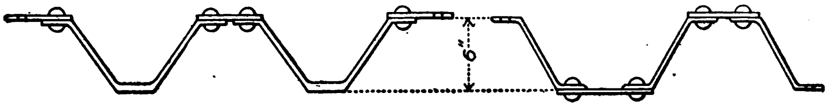


Fig. 61. Fig. 62.

quiring twice as much riveting as the former, half of it being in the bottom of the troughs.

The triangular form in Fig. 63 has been used to a limited extent, the lower edge not being well adapted for support, while the inclined sides render web connections with the girders less convenient. Fig. 65 shows a modification of the rectangular trough in which Z-bars take the place of the angles and vertical plates of Fig. 64.

The requirements of track elevation in large cities for very shallow floors in the plate-girder bridges at street crossings have led to several designs of floors composed of I-beams and continuous cover plates. In some cases the transverse I-beams

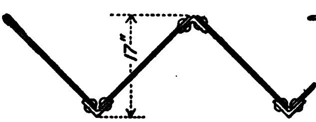


Fig. 63.

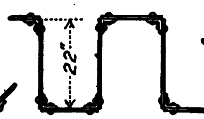


Fig. 64.

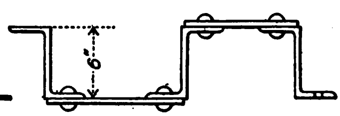


Fig. 65.

are spaced so close together as not to require any stringers, while in others they are spaced somewhat farther apart and short stringers are used to support the rails.

In highway bridges those floors are called solid floors in which there is either a continuous metal floor, or in which the metal and some other material, like concrete, for example, form

the permanent foundation to receive some kind of street and sidewalk surface paving.

The form most generally used consists of buckled plates supported on stringers and floor beams properly spaced, upon which concrete is placed to receive the rest of the pavement. Buckled plates were introduced in this country in 1875. Plain and corrugated or arched plates are also used in a similar manner, while some of the types of trough floors employed in railroad bridges have been adopted for highway bridges where shallow floors were required.

The great lateral strength of all solid floors renders a lateral system for them unnecessary.

In the next article are given references to periodicals relating to the description, history, and development of solid floors in bridges, together with a few examples, in which the applications described apply to more than one class of bridges. In Art. 48 are given a considerable number of references, arranged chronologically, to the various types of solid floors used in plate-girder bridges in this country, the knowledge of whose details will repay careful study.

ART. 47. SOLID BRIDGE FLOORS — REFERENCES.

Solid Bridge Floors, Railroad Gazette, vol. 22, pages 449 and 476, June 27 and July 4, 1890.

On the introduction of solid floors into America; dimensions and weights of splayed channel sections; illustrations showing different types of construction for troughs with these sections; and methods of protecting the metal against corrosion.

Thin Floors for Bridges. By A. F. ROBINSON. Trans. Am. Soc. C. E., vol. 27, pages 483 and 680, Nov. and Dec., 1892.

Requirements for shallow floors for which there is an increasing demand; proposed design of a rectangular trough floor with its connections for through pin and plate-girder bridges; detail drawings; reasons for adopting the rec-

tangular section; deflections of the solid floor in the Clyde Viaduct, under various speeds of locomotive used in tests; life of floor, weight and cost; discussions giving practical experience; and sketches of several forms of trough floors.

Experience with Solid Metal Bridge Flooring. By G. B. FRANCIS. Engineering News, vol. 32, page 399, Nov. 15, 1894.

Brief discussion of the relative value of several types of floors and of their connections in through and deck bridges; reference to cases where wooden floors answer the requirements.

Concrete and Iron Bridge Floors. By FR. VON EMPERGER. Engineering News, vol. 33, page 106, Feb. 14, 1895.

Note on the difference in cost between the Melan system of construction and a floor with corrugated plate and concrete.

The Terminal Improvements at Providence. Railroad Gazette, vol. 27, page 457, July 12, 1895.

General description of the deck plate-girder bridges, with metal trough floors, over Gaspee, Francis, and Promenade streets, and of the deck pin bridge, with ballast-covered wooden floor, over Woonasquatucket river; views of the trough flooring and bridges.

Solid Floors for Railroad Bridges. Their Merits and the Calculation of their Stresses. By HENRY GOLDMARK. Journal of the Association of Engineering Societies, vol. 15, page 69, Aug., 1895.

Severe requirements of bridge floors; determination of the distribution of concentrated loads in two types of trough floors by the method of least work; discussion including design for floor of closely spaced transverse I-beams, with or without continuous deck plates.

Removal of Grade Crossings in Buffalo, N. Y. Engineering News, vol. 34, page 266, Oct. 24, 1895.

History of the grade-crossing movement; drawings showing the details of two types of solid floors in beam, plate-girder, and riveted truss highway bridges.

Solid Floor Bridges for Railroads and Highways. By F. C. OSBORN. Jour. Assoc. Eng. Soc., vol. 15, page 232, Dec., 1895.

Historical sketch of the development of solid metal floors for railroad and highway bridges in England and America, containing illustrations of the various forms of section designed, and naming the bridges in which they were first used respectively. The descriptions include several designs of floors composed of I-beams and plates.

New Solid Floor for Highway Bridges. Engineering News, vol. 36, page 18, July 9, 1896.

Triangular troughs of corrugated and galvanized sheet iron or steel.

The Bridge Work of the Pittsburg, Bessemer, and Lake Erie, and Union Railways. Engineering News, vol. 44, page 102, Aug. 16, 1900; or, Special Bridge and Viaduct Construction in Western Pennsylvania. Engineering Record, vol. 41, page 465, May 19, 1900.

Description and drawings of fire-proof floor for the hot-metal track on the Union Railroad.

The following references are to solid floors in beam bridges:

Solid Bridge Floors. Engineering News, vol. 22, page 465, Nov. 16, 1889.

Drawings showing two bridges with splayed channel and rectangular troughs respectively. Ties in broken stone ballast.

The Sacramento Avenue Subway, Chicago, Ill. Engineering News, vol. 30, page 228, Sept. 21, 1893.

The floor consists of concrete arches supported between I-beams spaced two feet apart.

Railway Bridges of Short Span. By F. W. WILSON. Engineering News, vol. 35, page 299, May 7, 1896.

The drawings show sections of bridges in which the ballast is supported respectively by track rails and by rectangular troughs. The details are given of another type in which longitudinal I-beams support transverse I-beams covered by a continuous flat plate on which the rails rest in continuous channels.

Concrete and Expanded Metal Highway Bridge Construction in Allegheny County, Pa. Engineering News, vol. 41, page 50, Jan. 26, 1899.

The floor consists of arches of concrete and expanded metal supported by I-beams.

Economical Steel and Masonry Highway Bridge Work at Rye, N. Y. Engineering Record, vol. 44, page 412, Dec. 13, 1900.

Longitudinal I-beams support shallow transverse I-beams, spaced 26 inches apart, which carry a concrete foundation for the paved surface of roadway and sidewalks.

ART. 48. SOLID FLOORS IN PLATE-GIRDER BRIDGES—REFERENCES.

The following references relate to descriptions and illustrations of various types of solid floors in plate-girder bridges for both railroads and highways.

Solid Floor Plate Girder, New York Central and Hudson River Railroad. Engineering News, vol. 22, page 488, Nov. 23, 1889.

General drawing of a double-track deck bridge with a clear span of 20 feet. Notes on the effect of solid floors on the cost of masonry and of maintenance. Trough floor with splayed channel plates.

A New Style of Iron and Ballast Bridge Floor. Engineering News, vol. 28, page 386, Oct. 27, 1892.

Detail drawing of one end of a through girder with triangular trough floor on the New York, Providence and Boston Railroad. Span, 93′ 0″.

A Double Expansion Device for Bridges. Engineering News, vol. 30, page 140, Aug. 17, 1893.

Drawings of partial section and side elevation of a four-track through bridge on the Lake Shore and Michigan Southern Railroad. The splayed channel troughs are placed longitudinally and rest on top of the floor beams.

The Ninety-Third Street Subway under the Illinois Central Railroad Tracks. Railroad Gazette, vol. 26, page 228, Mar. 30, 1894.

Drawings show some details of an eight-track through skew bridge in Chicago. Length of girders, 36′ 6″. The trough floor of splayed channels supports the rails without ballast or cross-ties.

Solid Floor System, Ben Venue Bridge, Pittsburg, Pa. Engineering News, vol. 32, page 184, Sept. 6, 1894.

Illustration of partial section of the asphalt and concrete paving on a continuous corrugated plate floor which is supported by the stringers and floor beams of a deck girder bridge.

The Archer Avenue Subway, Chicago, Ill. Engineering News, vol. 32, page 291, Oct. 11, 1894; or, Railroad Gazette, vol. 26, page 718, Oct. 19, 1894.

Drawings showing details of the rectangular trough floor which is connected to the web without interfering with the chord rivets, and so that the troughs are even with the under side of the girder flanges. The rails rest on rail plates riveted to the troughs. No ballast.

A New Solid Metallic Bridge Floor. Engineering News, vol. 32, page 69, July 26, 1894. Solid Floor for Gaspee and Promenade Street Bridges, Providence, R. I. Engineering News, vol. 32, page 401, Nov. 15, 1894.

Conditions which led to the design of this new trough section (Fig. 60) and the merits claimed for it. Views show the trough floor attached to the webs of deck girders in a bridge of great width.

The Gaspee Street Bridge, Providence, R. I. Engineering Record, vol. 31, page 130, Jan. 19, 1895.

Detail drawings of the outer through girder (span, 76′ 8½″) and of the intermediate deck box girders (span, 49′ 11″), and sketches showing the connections of the trough floor which is supported on shelf angles. View of the floor in position. Loading, impact allowance, and unit stresses. Arrangement of floor permits shifting of railroad tracks.

New Elevated Structure of the St. Louis Terminal Railroad Association. Railroad Gazette, vol. 27, page 162, Mar. 15, 1895.

Sections showing details of buckle-plate floor construction. The buckle plates are supported by the stringers and by shelf angles on the through girders of the double-track structure.

Bridges for the Chicago Track Elevation. Railroad Gazette, vol. 27, page 178, Mar. 22, 1895; or, Elevated Main Line Railway Structure in Chicago. Engineering Record, vol. 31, page 310, Mar. 30, 1895.

Detail drawings of the solid floor on the through girder bridges of the L. S. & M. S. and C., R. I., & P. Railroads, and partial plans of the girders whose clear span is 66 feet. The floor consists of a continuous flat steel plate supported by transverse 10″ I-beams fastened to the girders by hanger plates. The surface plate is also riveted to the girders by connecting angles. The rail plates are laid directly on the floor plates.

Solid Bridge Floors of Old Rails. Railroad Gazette, vol. 27, page 770, Nov. 22, 1895; or, Engineering News, vol. 35, page 230, Apr. 2, 1896.

Sketches showing the arrangement of continuous floors of old rails supporting ballast used in deck girder bridges on the Chesapeake and Ohio Railroad.

Track Elevation, Chicago and Northwestern Railway, in Chicago. Engineering News, vol. 36, page 114, Aug. 20, 1896; or, Railroad Gazette, vol. 28, pages 548 and 565, Aug. 7 and 14, 1896.

Detail drawing of through plate-girder bridge with a clear span of 66 feet, details of solid floor and track construction, and views of bridges during erection. The floor beams, spaced 5 feet apart, are built up of two 10″ channels with filler web plate and cover plates. Each stringer consists of a rectangular trough, containing an oak timber, 6″ × 16″, which supports the rail plates. The entire surface exclusive of that over the stringers is covered by flat plates riveted to stringers, floor beams, and girders.

Track Elevation in Chicago. Railroad Gazette, vol. 28, page 768, Nov. 6, 1896.

Partial elevation and section of a through bridge with a clear span of 66 feet, showing details of the solid floor of the same construction as that in the

Railroad Gazette, vol. 27, page 178, referred to above. The transverse I-beams are, however, 12 inches deep, and are dropped down even with the outer cover plate of the bottom flange.

The New Railway Terminals at Providence, R. I., Engineering News, vol. 37, page 59, Jan. 28, 1897.

Section of troughs (see Fig. 61) with dimensions of its elements. Two views of the steel flooring in place for the Francis Street bridge covering one and one-half acres. Most of it is deck construction.

Abolition of the Grade Crossings on the Main Line of the Boston and Albany Railroad in Newton, Mass., III. Bridges over Railway Tracks. By W. G. S. CHAMBERLAIN. Jour. Assoc. Eng. Soc., vol. 21, page 62, Aug., 1898.

Roadway floor of brick arches between stringers covered with concrete, binder, and asphalt. In sidewalks curved plates are used instead of brick arches.

Some Short-span Railway Bridges. Engineering Record, vol. 40, pages 6 and 71, June 3 and 24, 1899.

Drawings and descriptions of three types of standard ballast floors on the Chicago and Northwestern Railroad. A double-track deck bridge with a clear span of 38 feet has a trough floor of splayed channel section. A through bridge for three tracks and a sidewalk, the span being about 71 feet, has side stringers 12′ 2″ apart, connected by gusset plates to girders and supporting by web connections the shallow box floor beams, spaced 5′ 1½″, and the transverse troughs. A double-track through bridge with span of 24 feet, has transverse 15″ I-beams, 43½″ apart, supporting buckle plates on the bottom flanges. Side channels support the ends of the buckle plates and confine the ballast.

The Eighteenth Street Bridge, Philadelphia. Engineering Record, vol. 40, page 451, Oct. 14, 1899.

Cross-sections show the longitudinal rectangular troughs of Z-bars and plates riveted on top of the floor beams, and the sidewalk brackets of a deck bridge whose main span is 52′ 2″. The bridge crosses the Philadelphia and Reading Railroad subway and has granolithic and asphalt paving on the sidewalks and roadway respectively.

A Plate-girder Highway Bridge with Concrete Floor. Engineering Record, vol. 42, page 323, Oct. 6, 1900.

Partial sections of the 65-foot center span of a deck bridge showing arches of concrete and expanded metal between transverse 15″ I-beams spaced 6′ 6″ apart. Asphalt paving. In the sidewalks 4″ I-beams, 4 feet apart, support the pavement.

ART. 49. PLATE-GIRDER BRIDGES — REFERENCES.

The following references relate to descriptions and illustrations more or less complete of plate-girder bridges constructed during the past decade. A careful study of these articles will familiarize the student with many important features of recent practice in plate-girder design and construction.

DECK RAILROAD BRIDGES.

Bridge Work on the Kansas City, Pittsburg, and Gulf Railroad. Engineering News, vol. 40, page 114, Aug. 25, 1898.

Spans of 50 and 60 feet. Brief description of the main details for these two standard lengths.

The Gaspee Street Bridge, Providence, R. I. Engineering Record, vol. 31, page 130, Jan. 19, 1895.

Span, 49′ 11″. Detail drawings of one of the intermediate box girders in this extremely wide bridge with solid floor.

Recent Small Bridges on the Baltimore and Ohio. Railroad Gazette, vol. 27, page 34, Jan. 18, 1895.

Span, 83 feet. General plan of the Turtle Creek bridge.

Plate Lattice Girder Bridge over Little Missouri River, Northern Pacific Railway. Engineering News, vol. 38, page 45, July 15, 1897.

Span 100 feet. Completely dimensioned detail drawings and three half-tone views. The ends have solid web plates while the middle portion is like a riveted truss.

Northern Pacific Standard Bridge Plans. By RALPH MODJESKI. Journal of the Western Society of Engineers, vol. 7, page 51, Feb., 1901.

Spans of 60 and 100 feet. Complete detail drawings.

Two Long-span Plate-girder Bridges. Engineering News, vol. 27, page 316, April 2, 1892.

Span about 100½ feet. Partial detail drawings of the Beaver river bridge on the Pittsburg and Lake Erie Railroad. One of the earliest plate-girder bridges to be constructed with a camber.

Long-span Plate-girder Bridge. Engineering Record, vol. 39, page 140, Jan. 14, 1899.

Span, 102′ 1″. Brief description, some computations, and drawings showing some of the details of a Pennsylvania railroad bridge at Bridgeport, O.

The Janesville Bridge. Engineering Record, vol. 44, page 6, July 6, 1901.

Span, 114½ feet. Description of the principal details of a bridge on the Chicago, Milwaukee, and St. Paul Railway at Janesville, Wis. Drawing of the expansion bearings.

THROUGH RAILROAD BRIDGES.

A Thin Floor Plate-girder Bridge. Engineering News, vol. 27, page 345, April 9, 1892.

Span, 53′ 2″. Double-track bridge with three girders, on the Chicago and Northwestern Railroad. Elevation of the middle and outside girders and some details of the floor system. The floor beams are only about 2½ feet apart.

Bridges for the Chicago Track Elevation. Railroad Gazette, vol. 27, page 178, March 22, 1895.

Span, 68′3″. Partial plan, elevation, and cross-section of the four-track (5 girders) bridges used at many 66-foot street crossings by the Lake Shore and Michigan Southern, and the Chicago, Rock Island and Pacific Railroads.

Track Elevation in Chicago. Railroad Gazette, vol. 28, page 768, Nov. 6, 1896.

Span, 68′3″. Outline elevation and section of the six-track bridge at Garfield Boulevard. Partial elevation and section showing details.

Track Elevation, Chicago and Northwestern Railway, in Chicago. Engineering News, vol. 36, page 114, Aug. 20, 1896; or, Railroad Gazette, vol. 28, pages 548 and 565, Aug. 7, 14, 1896.

Span, 69 feet. Partial elevation, plan and section showing details. A number of half-tone views.

The Gaspee Street Bridge, Providence, R. I. Engineering Record, vol. 31, page 130, Jan. 19, 1895.

Span, 76′ 8½″. Detail drawings of the end girders of the twelve-track bridge on the New York, New Haven and Hartford Railroad.

A Seventy-one Ton Plate Girder. Engineering Record, vol. 42, page 318, Oct. 6, 1900.

Span, 83′ 3½″. Description of the details of the middle girder of the four-track bridge on the New York Central and Hudson River Railroad at Oriskany, N. Y. See Figs. 49 and 50, Art. 44.

Standard Designs for Girder Bridges, Northern Pacific Railway. Engineering News, vol. 38.

Span, 85 feet. Detail drawings of a plate-lattice girder.

Bridge Work on the Baltimore and Ohio Railroad. Engineering Record, vol. 41, page 271, March 24, 1900.

Description of the principal features of recent construction in replacing old bridges. Some girders are 95 feet long. Two half-tone views show some of the details.

Two Long-span Plate-girder Bridges. Engineering News, vol. 27, page 316, Apr. 2, 1892.

Span, 99′ 9″. Partial detail drawings of the Mattabesset river bridge on the New York, New Haven and Hartford Railroad.

Standard Plans for 100-Foot Through Plate-lattice Girder Bridges, Northern Pacific Railway. Engineering News, vol. 38, page 23, July 8, 1897.

Span, 100 feet. Complete detail drawings and three half-tone views.

Northern Pacific Standard Bridge Plans. By RALPH MODJESKI. Journal of the Western Society of Engineers, vol. X,6 page 51, Feb., 1901.

Spans of 60 and 100 feet. Complete detail drawings.

A 103-Ton Plate Girder. Engineering Record, vol. 43, page 102, Feb. 2, 1801.

Span, 107′ 8″. Full description of the details of the middle girder of a four-track bridge on the New York Central and Hudson River Railroad over the Clyde river, east of Lyons, N. Y. Outline drawings of parts of the structure. See Figs. 42, 48, and 58, Arts. 42, 44, and 45.

A Long Double-track Plate-girder Span. Engineering Record, vol. 40, page 474, Oct. 21, 1899.

Span, 113½ feet. Full description of the details of the Maiden Lane bridge of the New York Central and Hudson River Railroad at Albany, N. Y.

A 128-Foot Plate-girder Railroad Bridge. Engineering Record, vol. 41, page 565, June 16, 1900.

Span, 125′ 2½″. Description of the principal details of bridge No. 7 on the Bradford division of the Erie Railroad. The drawings show an outline elevation with shears and flange stresses, and a few details. See Figs. 41, 52, and 57, Arts. 41, 44, and 45.

DECK HIGHWAY BRIDGES.

The Spring Avenue Bridge, Troy, N. Y. Engineering Record, vol. 32, page 471, Nov. 30, 1895.

Span, 102 feet. The 36″ roadway and two 12″ sidewalks are supported by three girders, spaced 22′ apart. The pavement is laid on a buckle-plate floor. A half section near the center of the bridge and an elevation and sections of the end of one girder supplement the description.

THROUGH HIGHWAY BRIDGES.

The Wellsville Over-grade Bridge. Engineering Record, vol. 42, page 101, Aug. 4, 1900.

Span, 47 feet. This bridge carries Madison St. in Wellsville, N. Y., over the Erie Railroad tracks. The girders are 19 feet apart, and the floor beams

dropped down in the clearances between the tracks so as to make the distance from the top of the railroad clearance to the street surface only one foot. An outline elevation, plan, and section together with dimensioned details of floor connections are given.

Plate-girder Park Foot Bridge at Madison, N. J. Engineering News, vol. 44, page 134, Aug. 23, 1900.

Clear span, 50 feet. The girders are 10 feet apart, the upper flanges inclined from the ends to the middle and the lower flanges curved so as to resemble an arch. An outline elevation and longitudinal section, and a half-tone view of the bridge are shown. No details.

Plate-girder Highway Bridge, Brookline, Mass. Engineering News, vol. 26, page 257, Sept. 19, 1891.

Clear span, 55′ 2½″. Carries the Dean road over the Boston and Albany Railroad tracks. Roadway, 33′; and two sidewalks, each 9′ 6″ wide. Upper flanges of girders curved throughout. The text gives an abstract of the specifications, and the drawings show the details and dimensions.

The State Street Plate-girder Bridge. Engineering News, vol. 27, page 205, Feb. 27, 1892.

Span, 86 feet. Bridge across the Rock River at Rockford, Ill. The girders are 40 feet apart and the 10-foot sidewalks are supported on cantilever brackets. The details of the girders and floor system are shown on the drawings, which also include an outline elevation and plan of the bridge without the approaches.

Transporting a Plate Girder 123 Feet Long. Engineering News, vol. 34, page 174, Sept. 12, 1895.

Span, 119′ 10½″. General elevation and plan of one girder and a few details of the solid floor and expansion joints.

CHAPTER VII.

DESIGN OF A PLATE-GIRDER BRIDGE.

ART. 50. SPECIFICATIONS.

Let it be required to design a deck plate-girder bridge for a single-track railroad, the span being 80 feet between centers of supports. The bridge is to be located on a straight track, and its material, with the exception of the rivets and the track, is to be medium steel.

In order to avoid the inconvenience to the student of continually referring to other pages while following the computations required for the design, the specifications will not be printed separately, but will be given in the text as needed. That the

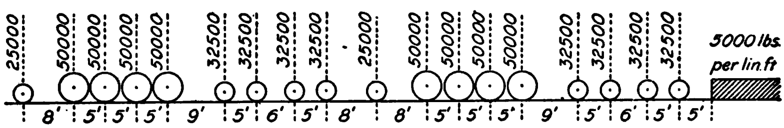


Fig. 66.

student may gain some familiarity with the range of variation in modern specifications relating to some of the details, according to a number of the leading standard specifications for steel railroad bridges, references will sometimes be made to such variations before adopting the respective requirements.

The live load is to consist of two consolidation locomotives and a train, as shown in Fig. 66, or an alternative load of 120 000 pounds, equally distributed on two pairs of driving wheels, spaced 6 feet center to center. The above loading is known as COOPER'S Standard, Class E 50, and equals the 1900 standard of the Lehigh

Valley Railroad for its main line, with the exception of the alternative load, which is slightly different. It is also the 1901 standard of the Baltimore and Ohio Railroad.

The allowance for impact due to the live load is that recommended by WADDELL, the coefficient of impact being

$$I = 400/(L + 500),$$

in which L is the length in feet of that portion of the span which is covered by the live load when the maximum stress under consideration is produced, and I is the percentage by which the maximum static stress is to be increased.

ART. 51. DEPTH AND SPACING.

A comparison of a number of recent designs of plate girders for some of the principal railroads shows a variation in the depth from one-tenth to one-twelfth of the span. In one case, where the span is over 125 feet, the depth is only one-fourteenth of the span; and in a few others the limitations imposed by track elevation led to the adoption of the same relative depth. There are also a few cases of very short spans in which the depth is one-eighth or one-ninth of the span. The average ratio of the depth to the span is found to be the reciprocal of 10.5.

Some specifications state that the depth of a plate girder shall preferably be not less than one-tenth of the span, and others substitute the fraction of one-twelfth, while many make no reference to the subject.

An interesting formula, deduced by HENRY SZLAPKA, for finding the economic depth of plate girders, was published in Engineering Record, vol. 35, page 363, March 27, 1879. The economic depth was found to vary from one-seventh to one-ninth of the span under the conditions and assumptions mentioned in the article.

The spacing of deck girders, center to center, ranges from 5 to 9 feet. Comparatively few are spaced less than 6½ feet apart, and the larger girders are spaced in proportion to their depth. WADDELL'S rule makes it the nearest even foot to one-tenth of the span. Although in some deep girders the spacing slightly exceeds the depth, in general it is somewhat less, the difference rarely exceeding 6 inches. For the design in this chapter let the depth of the web plate be taken as 7 feet and the spacing of the girders as 8 feet. A discussion of the economic depth may be found in Art. 66.

Through girders must be spaced so as to provide the necessary clearance specified, although in track elevation, where the bridges must carry many tracks at the standard distance of 13 feet apart between center lines, they have been spaced at the same distance. The usual spacing ranges from about 14½ feet for short spans to 17 feet for long spans for a single-track bridge.

ART. 52. THE WOODEN FLOOR.

The floor usually consists of cross-ties of rectangular section, notched at least ½ inch over the flange of each supporting girder, certain ties being secured to each flange by a ¾-inch hook bolt. Practice varies by making this attachment apply respectively to every tie, to alternate ties, or to every third, fourth, or fifth tie. The space between cross-ties is not generally to exceed 6 inches nor to be less than 5 inches.

Outer guard timbers 6 by 8 inches are laid flat and parallel to the track rails and notched one inch over every cross-tie, to hold them in their relative positions longitudinally. Their attachment to the cross-ties by ¾-inch bolts varies in practice in the same manner as that noted above for securing the ties to the girders. The guard timbers are spliced over a tie by a half-lap joint 6 inches long, and a bolt must be passed through the

splice to secure the ends of both timbers to the tie. The inner face of the guard timber is placed anywhere between 11 inches from the gage side of the rail head to 5 feet 4 inches from the center of the track. It should be placed near the end of the standard 12-foot tie.

Sometimes inner guard timbers are placed with a clearance of 6 to 10 inches between them and the rail heads, either with or without angle irons to protect the outer edges, but more frequently old track rails are employed as the inner or true guard rails.

The alternative loading given in Art. 50 causes the greatest stress in the cross-ties. The load on one wheel is 30 000 pounds, and it is customary to regard this load as distributed over three cross-ties. The live load for one tie is therefore 10 000 pounds and the impact 8000 pounds. Assuming the weight of the track at 450 pounds per linear foot, and that one tie will carry a length of track of $1\frac{1}{4}$ feet, it will be sufficiently precise to regard this entire load as concentrated at the track rails. The cross-tie then acts as a beam whose supports are 96 inches apart and carrying two equal and symmetrically placed concentrated loads $59\frac{1}{2}$ inches apart, each of which is 18 280 pounds.

For a long, leaf-yellow pine cross-tie a unit stress of 2000 pounds per square inch in the outer fiber may be taken when the effect of impact is considered. If b be the breadth, and d the depth of the cross-tie, and the bending moment is equated to the resisting moment, both being expressed in pound-inches, there follows

$$333\,600 = 2000\, bd^2/6,$$

whence $bd^2 = 1000$. To determine the breadth, let the safe bearing on the side of the fiber be taken at 400 pounds per square inch. The bearing area required is then $18\,280/400 = 45.7$ square inches, and if the width of the base of the rail

be 6 inches, the breadth b must be 8 inches. The net depth is therefore 11.2 inches, making the gross depth 11.7 or 12 inches. Since the notch in the cross-tie is so near the section under the track rail, only the net depth can be used in computing the strength under flexure. On account of the variation in the total thickness of cover plates, cross-ties 13 inches deep may be required near the ends of the girder.

Let the cross-ties be spaced 6 inches in the clear, and every alternate one bolted to the girder flange and the wooden guard rail respectively. At $3\frac{3}{4}$ pounds per foot, board measure, the weight of one tie 12 feet long is 360 pounds. This makes the weight of the cross-ties 310 pounds per linear foot. The weight of rails, splices, guard rails, bolts, spikes, etc., will be assumed at 160 pounds per linear foot.

ART. 53. WEB SECTION.

SPECIFICATION. — The rivets used shall be seven-eighths of an inch in diameter. The shearing stress in web plates shall not exceed 12 000 pounds per square inch; but no web plate shall be less than $\frac{3}{8}$ inch in thickness.

According to the method explained in Part II, Art. 43, the maximum live-load shear is found to be 155 100 pounds, and from the formula for the coefficient of impact given in this chapter, Art. 50, the corresponding impact allowance is 107 000 pounds.

The net weight of one girder and one-half of the lateral bracing and cross-frames will be assumed as 45 200 pounds, which, with the weight of track at 235 pounds per linear foot for each girder, makes the dead load 64 000 pounds, and the dead-load shear 32 000 pounds.

The total vertical shear at the support is then 294 100 pounds, and for a specified unit shearing stress of 12 000 pounds per square inch the net section of the web must be

24.51 square inches, upon the assumption that the shear is uniformly distributed over the sectional area. The actual distribution of the shear is illustrated in Art. 59. The minimum thickness of $\frac{3}{8}$ inch allowed by the specification would permit only 14 rivets in the entire depth of 84 inches, and is hence insufficient. A thickness of $\frac{7}{16}$ inch will give a net depth of 56 inches, and if a diameter of 1 inch be deducted for each rivet hole in the web section, it will allow 28 rivets with an average pitch of 3 inches. This thickness will accordingly be used.

The standard rivet in plate-girder construction has a diameter of $\frac{7}{8}$ inch when cold, and the diameter of the rivet holes is made $\frac{1}{16}$ inch larger, so that the heated rivet can be readily inserted. All specifications agree in requiring deductions to be made for rivet holes in tension members with diameters assumed to be $\frac{1}{8}$ inch larger than that of the rivet before driving, but no reference is made to the corresponding deduction in members subject to shear.

It may be added that with the rapidly increasing practice of punching the holes smaller than the rivet diameter and then reaming the holes after the parts are assembled, or of drilling the holes in the solid, the reason which originally led to the deduction of such a large excess for rivet holes is fast disappearing. When these methods of forming the rivet holes are adopted, a clause might well be added in the specification prescribing the use of the actual net section in the computations.

ART. 54. SECTIONAL AREA OF FLANGES.

Since a plate girder under the action of vertical loads is a beam, the fundamental formula for flexure applies to it, namely,

$$M = \frac{SI}{c},$$

in which M is the bending moment at any section due to the

external forces, S the unit stress in the outer fiber whose distance from the neutral surface is c, and I the moment of inertia of the cross-section about the neutral axis.

In order to transform this equation so as to be convenient for the purpose of design, let t be the thickness and h the height or depth of the web plate, while A is the area of cross-section of each flange, and h_1 the distance between the centers of gravity of the flanges, usually called the effective depth of the girder. If the moment of inertia of each flange about its own neutral axis be neglected, as it is relatively small, the following expression may be written for the moment of inertia of the girder:

$$I = 2A\left(\frac{h_1}{2}\right)^2 + \frac{th^3}{12} = \frac{Mc}{S}.$$

Substituting the value of c which is very nearly equal to $\frac{1}{2}h$, and transposing,

$$A = \frac{Mh}{Sh_1^2} - \frac{th^3}{6h_1^2}.$$

But h_1^2/h is approximately equal to h_1, and h^3/h_1^2 is approximately equal to h, whence

$$A = \frac{M}{Sh_1} - \frac{th}{6}; \qquad (1)$$

that is, if the plate girder had solid sections throughout, the area of each flange would be less than that required to resist the entire bending moment, by an amount equal to one-sixth of the section of the web plate.

Although the gross area of the upper half of the girder section may be employed, since it is under compression, only the net section of the lower half should be used. The method therefore adopted is to design the lower flange, which is subject to tension, and then to make the upper flange of the same gross sectional area as the lower one. In applying the

above formula (1), the last term is interpreted as meaning one-sixth of the net section of the web plate. When stiffeners or splices are located at or near the section under consideration, the net section differs considerably from the gross section.

If one-inch rivet holes be deducted for $\frac{7}{8}$-inch rivets, one-sixth of the net section of the web plate becomes 10.3 percent of the gross section when the rivets in the vertical section have the minimum allowable pitch of three diameters or $2\frac{5}{8}$ inches; 11.1 percent for a pitch of 3 inches; 11.9 percent for a pitch of $3\frac{1}{2}$ inches; and 12.5 percent for a pitch of 4 inches, which would rarely be exceeded. If, however, the deduction be made for the actual rivet holes of fifteen-sixteenths of an inch in diameter, the respective percentages will be 11.5 for a pitch of 3 inches, 12.2 for a pitch of $3\frac{1}{2}$ inches, and 12.8 for a pitch of 4 inches.

Some recent specifications state that one-eighth of the gross area of the web plate is to be regarded as effective flange area, but the preceding paragraph shows that this allowance is in many cases somewhat too large. It is not only desirable on theoretical grounds that the resistance of the web plate to flexure should be considered in the design, but also for the practical reason that it encourages the use of thicker web plates and a greater depth where this is not otherwise limited, thereby increasing the life as well as the stiffness of the structure.

Turning now to the design under consideration, the bending moments due to the live load specified in Art. 50 are found at sections five feet apart, according to the method described in Part II, Art. 42. The absolute maximum moment due to this load is found to be 2 703 000 pound-feet at a section 0.2 foot from the center, where the moment is practically the same. The allowance for impact is 1 864 000, and the dead-load

moment is 640 000 pound-feet, making the total bending moment 5 287 000 pound-feet, or 63 444 000 pound-inches.

Placing the backs of the flange angles one-eighth inch beyond the edges of the web plate and assuming the centers of gravity of the flanges to be 1.5 inches less than the distance back to back of angles, the approximate effective depth is $84 + 0.25 - 1.5 = 82.75$ inches. For a specified unit tensile stress of 17 000 pounds per square inch, and assuming 12 percent of the gross web section as effective flange area, the required net area of the lower flange,

$$A = \frac{63\,444\,000}{17\,000 \times 82.75} - \left(0.12 \times \frac{7}{16} \times 84\right) = 45.10 - 4.41 = 40.69,$$

the result being expressed in square inches.

ART. 55. COMPOSITION OF THE FLANGES.

SPECIFICATION.—About one-half of the flange section shall consist of angles, or else the heaviest sections of angles must be used, and the number of cover plates shall be as small as practicable. The cover plates shall be of equal thickness or decrease in thickness outward from the angles, and shall not extend more than four inches or eight times the thickness of the outside plate beyond the outer line of rivets. The net section of the tension flange shall be determined by a plane cutting it square across at any point, and the greatest number of rivet holes which can be cut by any such plane, or whose centers come nearer to it than two and a half inches, are to be deducted from the gross section in computing the net area. The compression flange shall have the same gross section as the tension flange.

The effective diameter of any rivet shall be assumed the same as its diameter before driving; but in making deductions for rivet holes in tension members, the diameter of the holes shall be assumed to be one-eighth of an inch larger than that of the rivet.

One-half of the net flange area, determined in the preceding article, is 20.35 square inches, and hence either $8'' \times 6''$ or $8'' \times 8''$ angles are required. Adopting the latter and observing the rest of the above specifications, the flange may be made up as follows (see Fig. 67):

2 angles, $8'' \times 8'' \times \frac{3}{4}''$; $2(11.44 - 1.50) = 19.88$ square inches.
3 cover plates, $18'' \times \frac{7}{16}''$; $3(7.88 - 0.88) = \underline{21.00}$
Total net section $= 40.88$ square inches.

Two rows of rivets will be required in each leg of the angles, the rivets in adjacent rows being staggered, and hence two rivet holes must be deducted from each angle and from each plate, the pitch of the rivets at the middle of the girder being certainly greater than $2\frac{1}{2}$ inches. For location of rivet lines see Art. 34.

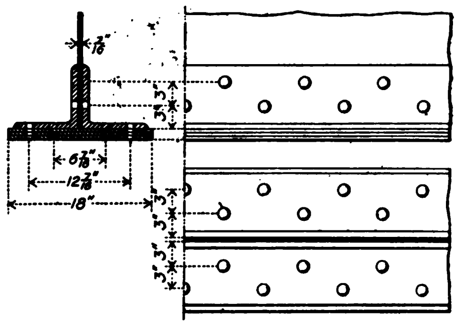


Fig. 67.

The center of gravity of this flange section is next computed and found to be 0.815 inch from the backs of the angles, while the corresponding distance for the gross section of the upper flange is 0.788 inch. The correct effective depth is, therefore, $84 + 0.25 - 1.60 = 82.65$ inches, which makes the revised flange area required equal to $45.15 - 4.41 = 40.74$ square inches. As this value does not exceed the net section given above, the composition of the flanges needs no revision.

If the section were moved so as to cut the adjacent rivets, the distance from the center of gravity of the net section of the lower flange to the backs of the angles would be reduced from 0.815 to 0.705 inch, and the effective depth of the girder increased to 82.76 inches. The average value is used by some designers.

In regard to the deduction of rivet holes for net section, one specification which is extensively adopted provides that the rupture of a riveted tension member is to be considered as

equally probable, either through a transverse line of rivet holes or through a diagonal line of rivet holes where the net section does not exceed by 30 percent the net section along the transverse line.

By comparing the revised and provisional flange areas the student may gain some idea as to the relative effect of small changes in the effective depth or in other items affecting it, and thus learn what degrees of precision are required in the various computations. As the actual sections of shapes are subject to slight variation and there are inaccuracies in workmanship, it is sufficient in practice to determine the effective depth to the nearest tenth of an inch.

While angles can be rolled of any thickness between the minimum and maximum given in the handbooks, the practice is quite extensive to use only standard angles whose thicknesses are expressed in full sixteenths of an inch.

ART. 56. WEB SPLICES.

SPECIFICATION. — Whenever practicable, plate girders shall be built without splices in the web, and when splices are necessary, their number shall be made as small as possible. The splice plates and rivets for the splices shall be such as to develop in every respect the full strength of the net section of the web, the main splice plates extending from flange to flange and having at least two rows of rivets on each side of the joints. In addition to these, two splice plates shall cover the vertical legs of the angles in each flange. The shearing stress on rivets shall not exceed 12 000 pounds per square inch of section, and the pressure upon the bearing surface of the projected semi-intrados (diameter times thickness) of the rivet hole shall not exceed 24 000 pounds per square inch.

According to the Carnegie handbook the extreme length to which a sheared steel plate 84 inches wide and $\frac{7}{16}$ inch thick is rolled is 380 inches. It will thus be possible to build a girder whose span is 80 feet with only two web splices.

In Art. 54 it was shown that one-sixth of the net section of the web is the equivalent flange area regarded as concentrated at the center of gravity of the flange, and which represents the share of the web in resisting the bending moment at any section. The web splice must accordingly be designed to transmit not only the shear in the section but also its proportionate part of the bending moment. This is accomplished with a sufficient degree of precision for all purposes of design when the splice plates and rivets are arranged to develop the full strength of the net section of the web to resist the bending moment only.

As not less than two rows of rivets are to be placed on each side of the joint, the rivets in the lower half of the splice will be arranged as indicated in Fig. 68, the pitch in each row when considered independently being 4 inches toward the neutral surface and 3 inches toward the flange, thus placing the rivets which are the most effective closer together. The numerals on the right of the figure represent the distances in inches from the rivets to the neutral surface.

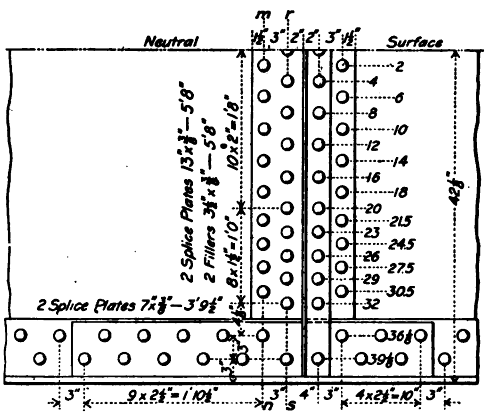


Fig. 68.

As the web is $\frac{7}{16}$ inch in thickness and 84 inches deep, and the unit stress in the outer fiber is 17 000 pounds per square inch, the resisting moment of the gross section is

$$\frac{1}{6} \times 17\,000 \times \frac{7}{16} \times 84 \times 84 = 8\,746\,500 \text{ pound-inches}$$

and that of the lower half is 4 373 300 pound-inches. To find the

resisting moment of the net section of the web it is necessary to deduct that of the diametral sections of the rivet holes in the outer row of the splice. Remembering that for members in tension the diameter of the rivet holes is to be taken as $\frac{1}{8}$ inch greater than that of the rivets before driving, the reduction of tensile stress in the web for a rivet hole at a distance from the neutral surface equal to that of the outer fiber of the web is $17\,000 \times 1 \times \frac{7}{16} = 7440$ pounds, while for one at a distance y from the neutral surface it is $(7440y/42)$ pounds. The moment of this stress about the neutral axis is $7440y^2/42$, and the sum of the moments for all the rivet holes in the row mn of Fig. 68 is $7440\,\Sigma y^2/42 = 7440 \times 4714/42 = 835\,100$ pound-inches. This leaves the resisting moment of the net section of the lower half of the web equal to 3 538 200 pound-inches.

With a unit stress of 24 000 pounds per square inch for the bearing on the side of the rivets, the allowable bearing of a $\frac{7}{8}$-inch rivet on the $\frac{7}{16}$-inch web plate is $24\,000 \times \frac{7}{8} \times \frac{7}{16} = 9190$ pounds. The combined strength of the splice plates must equal that of the web, but as no metal less than $\frac{3}{8}$ inch in thickness is allowed in good practice except for filling plates, $\frac{3}{8}$-inch splice plates will be used. Accordingly the bearing of the rivets in both splice plates combined is greater than that in the web. The rivets are in double shear and with a safe unit stress of 12 000 pounds per square inch, the value of a $\frac{7}{8}$-inch rivet in double shear is 14 430 pounds, and hence the bearing in the web governs the determination of the number of rivets in the splice. As the outer row of rivets is $39\frac{1}{8}$ inches from the neutral surface, the bearing value of a rivet at the distance y from the neutral surface is $9190y/39.125$, and the moment of the bearing is $9190y^2/39.125$. The sum of the moments of the bearing values of all the rivets in both rows mn and rs, exclusive of those in the flange, is

$$9190\,\Sigma y^2/39.125 = 9190 \times 7359/39.125 = 1\,728\,500 \text{ pound-inches.}$$

This result shows that twice as many rows of rivets would be required if no other splice plates were employed except those connecting that portion of the web which lies between the edges of the flange angles. Such an arrangement is not economical, since too large a proportion of the rivets are ineffective in resisting the bending moment. Let two splice plates $7'' \times \frac{3}{8}''$ be placed on the vertical legs of the flange angles, in order to connect those parts of the web plates which carry the highest unit stress. It is now required to find how many rivets, through these plates, are necessary to make the full strength of the splice rivets equal to that of the net section of the web plate. The resisting moment to be taken by the rivets through these longitudinal splice plates is $3\,538\,200 - 1\,728\,500 = 1\,809\,700$ pound-inches, and hence $\Sigma y^2 = 1\,809\,700 \times 39.125/9190 = 7705$ inches^2. Since the squares of 36.125 and 39.125 are 1305 and 1531 respectively, three rivets in each row are required, making $\Sigma y^2 = 8508$ inches^2, that is, six rivets are needed on each side of the joint as shown in Fig. 35. This number of rivets can transmit into the flange angles the full tensile strength of these splice plates, and hence the number of connecting rivets does not need to be increased on that account. However, as the rivets, through the vertical legs of the flanges, have the additional duty to transmit an increment of flange stress from the web plate to the angles, as will be explained in Art. 59, the plates must be extended toward the nearer end of the girder so as to contain enough rivets to take both of these stresses, the pitch being reduced to one-half the value that would otherwise be used. The exact length of the plates will be found in Art. 58.

If the vertical shear be taken into account, the bearing value of the outermost rivet is reduced from 9190 to 8640 pounds, since each rivet must take a shear of 3140 pounds. The allowable tensile stress in the outer fiber is also reduced from 17 000

to 15 800 pounds per square inch, since the total vertical shear at the section (12 feet from the middle) is 126 500 pounds, and the net section of the web is $(84-20)\frac{7}{16}=28$ square inches, the deduction being for 20 rivets, and the resulting unit shear 4520 pounds per square inch. (See Mechanics of Materials, Art. 75.) On introducing these values in the computations, Σy^2 is found to be 7533 instead of 7705 inches^2, thus requiring practically the same number of rivets. The former method, which is much simpler than the latter, is therefore sufficiently precise, as stated above.

The outer row of rivets in Fig. 68 is arranged so as to reduce the net section of the web as little as possible in that part which takes the greatest stress. The resisting moment of the net section of the web is 3 538 200/4 373 300 = 0.809 times that of the gross section, and hence one-sixth of this or 13.5 percent of the gross web area may be regarded as equivalent flange area. This slightly exceeds the value used in Art. 54; viz. 12 percent.

Art. 57. Web Stiffeners.

In a plate-girder web which consists only of a continuous web plate there exist at any point compressive and tensile stresses at right angles to each other whose magnitudes equal those of the vertical and horizontal shear at that point (Mechanics of Materials, Art. 119). The lines of the maximum compressive and tensile stresses cross each other at right angles at the neutral surface, and make angles of 45 degrees with that surface. The compressive stresses tend to buckle or wrinkle the web plate, while the tensile stresses tend to keep it straight.

Experience has led to the custom of stiffening the web by means of pairs of vertical angles placed on opposite sides of the web plate, and riveted together, whenever the clear distance between the flange angles is greater than about 50 times the

thickness of the web plate. In the specifications this ratio is given as 50, 60, or 64, or even as high as 80. These stiffeners are usually placed at distances apart not exceeding the depth of the girder, with a maximum limit of 5 or 6 feet, the former value being more generally specified.

Formerly the prevailing practice was to space them closer together toward the ends of the span as the shear increased, but at present the practice of spacing them at uniform intervals is very common. No rational theory has been developed upon which the design of intermediate stiffeners may be based.

It is not definitely known to what extent the addition of intermediate stiffeners modifies the distribution of stresses in the web plate of the girder. Although a few experiments have been made which, together with observations in the maintenance of girders under traffic, throw some light on the subject, it has not received the investigation which its importance seems to demand. Some of these facts are recorded in engineering periodicals, to which references are given at the end of this article.

The intermediate stiffeners should be able to transmit to the web the heaviest concentrated load which may come upon it in a deck girder, or the greatest floor beam reaction in a through girder. In the example used in this chapter the greatest concentrated load is 30 000 pounds, and the addition for impact 24 000 pounds. With the specified compressive unit stress of 17 000 pounds per square inch, a sectional area of 3.18 square inches is required. The leg of the angle which is to be riveted to the web plate must not be less than $3\frac{1}{2}$ inches, as $\frac{7}{8}$-inch rivets should not be used in a smaller size (Art. 34). Two angles, $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$, the thickness being the least allowable, furnish much more than the necessary area, but their outstanding legs would not give an adequate support to the 8-inch flange angles which transmit the load to the stiffeners. The size will therefore be increased to $6'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$.

The end stiffeners must take the vertical shear from the web and carry it to the bearing plate. They do not act entirely as columns, since the load is distributed along the entire length. A somewhat lower working stress should be taken than that for simple compression,—say about 15 000 pounds per square inch. The sectional area of the end stiffeners must therefore be $294\,100/15\,000 = 19.61$ square inches. Six angles $6'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ will furnish an area of $6 \times 3.41 = 20.46$ square inches, and may hence be adopted. Sometimes it is specified that the projecting legs of all stiffener angles over the end bearings shall be as wide as the flange angles permit. This rule would accordingly require $7'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ angles.

All the stiffeners should be closely fitted to the upper flange angles in the deck girders, and the end stiffeners should also be fitted to the lower flange angles to secure a full bearing area. The lower ends of intermediate stiffeners in deck girders, and both of their ends in through girders, require merely a neat fit for the sake of appearance, since they transmit no stresses directly to the flanges. It should be added that many specifications make no distinctions in this respect. The number of rivets connecting the end angles to the web is $294\,100/9190 = 32$, since the bearing value of a $\frac{7}{8}$-inch rivet in a $\frac{7}{16}$-inch web is 9190 pounds. For the sake of uniformity, which simplifies construction, the same number of rivets will be used in all the stiffeners, and equal to that required in the inner rows of the web splice. This will give $3 \times 19 = 57$ rivets, without counting those which also pass through the flange angles, and whose duty is to carry stresses from the web into the flange.

In girders of double-track bridges the reaction of the support may be so large that a sufficient number of rivets cannot be put into the stiffeners. Instead of using wider angles to accommodate two rows of rivets each filler may be widened so as to take an extra row of rivets. The number of rivets then required in

the stiffeners alone is governed by their value in double shear. Sometimes a single plate on each side of the web replaces the separate fillers under the two or more pairs of end angles. See Plate I and Fig. 30.

The ends of plate girders should be finished with cover plates. In deck girders the corners are square, but in through girders the upper corner is generally rounded off to a radius which ranges from one-half to the full length of the bed plate of the support. The upper flange angles are usually cut just before reaching the curve and spliced to angles of reduced thickness which extend around the curve and down the ends, although sometimes the flange angles themselves are extended down to the support (Fig. 46, Art. 44).

The following articles give some idea of the character of the discussions which take place at intervals in regard to the stresses in the webs of plate girders and the function of stiffeners:

Specifications for the Strength of Iron Bridges. By Joseph M. Wilson, and discussion by W. H. Burr and E. Thacher. Transactions American Society Civil Engineers, vol. 15, pages 404, 430, 467, June, 1886.

Vertical or Inclined Stiffeners for Plate Girders. By C. A. P. Turner and J. B. Johnson, Engineering News, vol. 33, page 276, April, 1895. By C. A. P. Turner and J. P. Snow, Engineering News, vol. 33, page 339, May 23, 1895. By Henry Goldmark, Engineering News, vol. 34, page 43, July 18, 1895.

Thermal Condition of Iron and Steel under Stress, and Measurement of Stress by Means of Thermo-electricity. By C. A. P. Turner. Proceedings of the Engineers' Society of Western Pennsylvania, Sept., 1897. This paper contains the results obtained by tests of an experimental girder 10 feet long and 2½ feet deep. For a later valuable paper by Turner, see Proceedings of American Society of Civil Engineers, Jan., 1902.

Spacing Stiffeners in Plate Girders. By H. T. Beach, Engineering News, vol. 39, page 322, May 19, 1898. By Practical Bridge Builder, Engineering News, vol. 40, page 10, July 7, 1898. By Joseph M. Wilson and E. Marburg, Engineering News, vol. 40, pages 89 and 90, Aug. 11, 1898. By A. W. Buel, C. A. P. Turner, and Joseph M. Wilson, Engineering News, vol. 40, pages 154 and 155, Sept. 8, 1898. By F. G. Skinner and C. A. P. Turner, Engineering News, vol. 40, pages 339 and 400, Dec. 22, 1898. By H. T. Beach, Engineering News, vol. 41, page 106, Feb. 16, 1899.

Tests of the Stress in Plate-girder Stiffeners. By F. E. Turneaure. Engineering News, vol. 40, page 186, Sept. 23, 1898. This article contains the results of six measurements of the stresses in the stiffeners of a 75-foot plate girder under moving load.

Specifications for Steel Railroad Bridges. Discussion by George S. Morison and J. H. Worcester. Transactions American Society Civil Engineers, vol. 41, pages 184 and 193, June, 1899.

Proposed Specifications for Steel Railway Bridges. By J. W. Schaub, and discussion by H. E. Horton and Ralph Modjeski. Journal Western Society of Engineers, vol. 5, pages 355, 370, and 379, Oct., 1900.

A Direct Method of Spacing Rivets and Finding the Position, etc., of Stiffeners in Plate Girders. By E. Schmidt. With discussion. Transactions American Society Civil Engineers, vol. 44, page 550, June, 1901.

ART. 58. LENGTHS OF COVER PLATES.

In accordance with the requirement of some of the leading specifications one cover plate on each flange will be extended to the end of the girder. The others will be extended at each

end from 9 inches to a foot beyond the point where theory requires them in order to resist the maximum bending moments in the girder.

The combined net area of the two flange angles and one cover plate is 26.88 square inches, while the equivalent flange area of the web is 13.5 percent of its gross section (Art. 56) or 4.96 square inches, making the total flange area 31.84 square inches. The effective depth is found to be 80.9 inches, and hence the bending moment that may be resisted by this section of the girder is $17\,000 \times 31.84 \times 80.9/12 = 3\,649\,000$ pound-feet, and this is the value of the maximum moment at 17 feet from the support. This location is conveniently found by means of a diagram (Fig. 69) whose ordinates represent the maximum bending moments due to the live load, impact allowance, and dead load. Their values, expressed in kip-feet, are given in the following table, one kip being 1000 pounds:

MAXIMUM BENDING MOMENTS.

SECTIONS.	5'	10'	15'	20'	25'	30'	35'	40'
Live load	690	1253	1723	2067	2330	2523	2653	2703
Impact	476	864	1188	1426	1607	1740	1830	1864
Dead load	169	315	439	540	619	675	709	720
Total	1335	2432	3350	4033	4556	4938	5192	5287

The combined net area of the flange angles and two cover plates is 33.88 square inches, which added to the equivalent flange area of the web gives a total of 38.84 square inches. The effective depth is 81.9 inches, and the corresponding bending moment is $17\,000 \times 38.84 \times 81.9/12 = 4\,506\,000$ pound-feet, which is located at 24′ 6″ from the support. The approximate length of the outer cover plate will therefore be at least

$2(40'\ 0'' - 24'\ 6'') + 2 \times 9'' = 32$ feet 6 inches, and that of the second cover plate $2(40'\ 0'' - 17'\ 0'') + 2 \times 9'' = 47$ feet 6 inches. The exact lengths of the cover plates will be determined on the drawing after the rivets are located in the flanges, and the flange splices are located, for it is sometimes necessary to extend one cover plate to serve as a splice plate for another one.

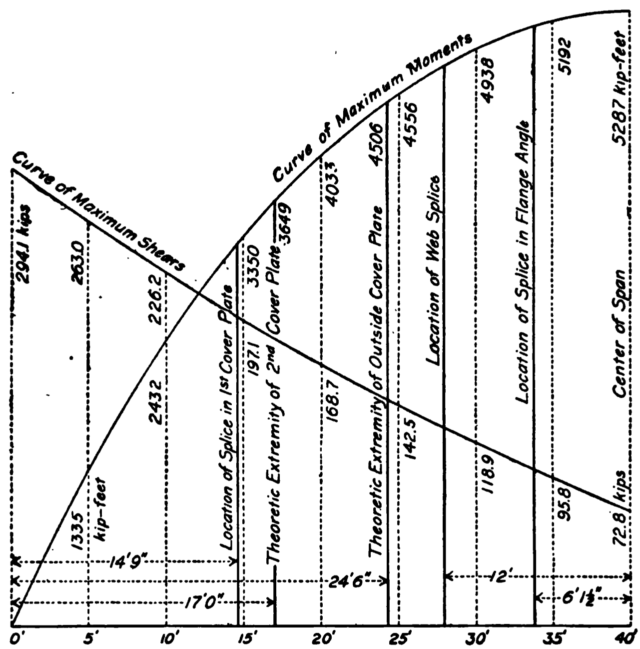


Fig. 69.

When the bending moments are determined by means of an equivalent uniform load, making the moment diagram a parabola, the lengths of cover plates may be quickly found either by the graphic method given by T. K. THOMSON in Engineering News, vol. 32, page 148, Aug. 23, 1894, or by the analytic method given by C. W. HUDSON in the same periodical, vol. 32, page 278, Oct. 4, 1894.

ART. 59. THEORETIC RIVET PITCH IN FLANGES.

The rivets uniting the web plate to the upper flange of a deck girder between any two given sections have two duties to perform: first, to transfer from the flanges to the web whatever load rests directly upon the flanges in this division; and, second, to transmit from the web to the flanges the increment of flange stress developed between these sections. The required number of rivets must then be such as to safely transmit these vertical and horizontal stresses when their resultant is a maximum. The horizontal component of the resultant is considerably greater than the vertical, except near the middle of the span, where in many cases the latter may be even greater than the former.

The maximum difference of flange stress between any two sections occurs when the difference between their respective bending moments is a maximum, provided the effective depth is the same. When the sections are taken a distance apart equal to dx, the difference of moments is dM, and if the entire bending moment were resisted by the flanges, the difference in flange stress would be dM/h_1, in which h_1 denotes the effective depth. The increment of flange stress per linear unit is then dM/h_1dx, which by mechanics equals V/h_1, the vertical shear being designated by V. This difference is a maximum when the maximum values of the vertical shear are inserted in the expression just given. Since the web plate, however, resists a part of the bending moment, V/h_1 must be multiplied by the ratio of the bending moment resisted by the flanges alone to the entire bending moment. This ratio equals that of the area of the flanges to the sum of the flange area and the equivalent flange area of the web as explained in Art. 54. The pitch of the rivets, or their spacing longitudinally, is then obtained on dividing the resistance of one rivet by the value just found. As the rivets connecting the web to the flanges are in double shear, the bearing value of a

rivet on the web plate will usually be less than the double shear, and therefore measures the strength of the rivet to be used in the computation.

The vertical shears whose determination was referred to in Art. 53 are given in the following table and expressed in kips, the sections being taken 5 feet apart. See also Fig. 69.

SECTION =	0'	5'	10'	15'	20'	25'	30'	35'	40'
Live load	155.1	139.9	120.6	104.3	89.6	76.3	64.6	53.3	42.1
Impact	107.0	95.1	81.6	72.8	63.1	54.2	46.3	38.5	30.7
Dead load	32.0	28.0	24.0	20.0	16.0	12.0	8.0	4.0	0.0
Total	294.1	263.0	226.2	197.1	168.7	142.5	118.9	95.8	72.8

The position of the live load which causes the maximum shears is such that the first driver of the locomotive (Art. 50) is just on the right of the section. Its weight on one rail and distributed over three ties, which, with the three spaces, cover 42 inches (Art. 52), is 25 000 pounds. The coefficient of impact for the shears varies from about 0.69 to 0.73, and using the average value 0.71 for this load, the impact allowance is 17 750 pounds. The corresponding weight of the track supported by one girder is 700 pounds, making a total load of 43 450 pounds, or 1035 pounds per linear inch.

At section 0′, which is at the support, each flange is composed of two angles and one cover plate (Art. 58), and according to the method given above the increment of flange stress per linear inch resisted by the flange alone is

$$\frac{26.88}{31.84} \cdot \frac{294\,100}{80.9} = 3069 \text{ pounds.}$$

The ratio 26.88/31.84 refers to the tension flange, having been used in Art. 58, but it may be applied to the compression flange as being sufficiently exact for this purpose.

The resultant of 3069 and 1035 pounds is 3236 pounds per linear inch, and the required pitch is $9190/3236 = 2.84$ inches, since the bearing value of a $\frac{7}{8}$-inch rivet in a $\frac{7}{16}$-inch web plate is 9190 pounds.

In a similar manner the pitch is determined at each of the sections, the results laid off as ordinates in Fig. 70, and a curve drawn through their extremities. This diagram gives the theoretic pitch at every point in the half span. The horizontal lines

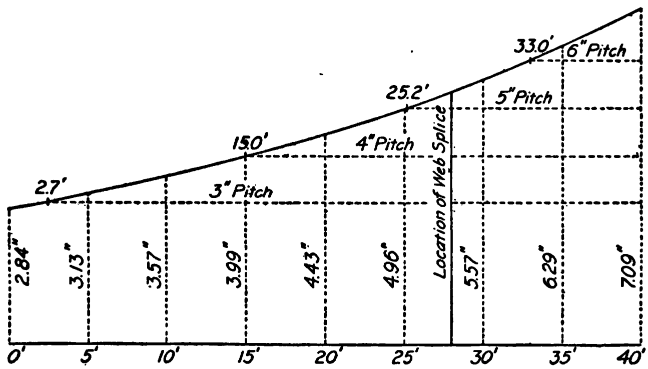


Fig. 70.

show that the pitch is 3 inches at 2.7 feet, 4 inches at 15 feet, 5 inches at 25.2 feet, and 6 inches at 33 feet from the center of the support.

If the vertical component of the rivet stress be neglected, the theoretic rivet pitch in the lower flange is obtained, that at the end of the girder being 2.99 inches, and at the middle 11.69 inches. A comparison of these values with those found for the upper flange indicates the direct influence of the load supported by the flange, on the pitch of the rivets.

The rivets connecting the angles and the cover plates must transmit that portion of the flange stress which is taken by the cover plates. These rivets are in single shear, and hence the strength of a rivet is measured by its value in single shear,

which is less than the bearing in either the cover plate or the angle. The value in single shear of a $\frac{7}{8}$-inch rivet at 12 000 pounds per square inch is 7220 pounds. The proportions of flange stress taken by one, two, and three cover plates respectively are approximately 26, 41, and 51 percent. The rivet pitches at the end of the girder, and at the points where the second and third (or outer) cover plates terminate theoretically (see Fig. 69, Art. 58), are found to be 9.1, 9.1, and 8.9 inches respectively.

For the purpose of comparison the rivet pitch at the end of the girder will also be determined by means of the horizontal shear. The direct effect of the vertical load on the flange rivets will be omitted in this comparison, as it does not affect the result.

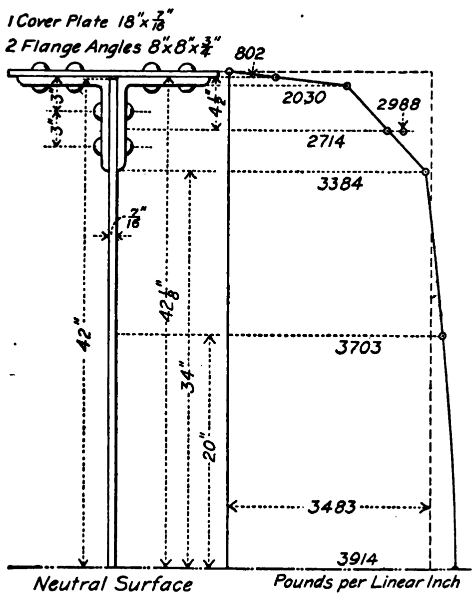


Fig. 71.

On the left of Fig. 71 is shown a vertical section of the upper half of the girder at the support, and on the right is a diagram whose abscissas represent the horizontal shear per linear inch, the values being computed (see Mechanics of Materials, Art. 78) upon the assumption that the cover plate, angles, and web plate are all parts of a single piece of solid metal. This assumed condition is evidently not equal in some respects to that of the girder, in which these parts are riveted together. The horizontal shear in a horizontal

section taken midway between the two lines of the flange rivets is thus found to be 2714 pounds per linear inch.

If the shear between the flange and the web be computed by substituting in the formula the moment of the entire section of both angles as well as of the cover plate, and omitting entirely any part of the web between the angles, the value of the horizontal shear per linear inch is 2988 pounds. This value is observed to be somewhat less than the horizontal shear in the web plate directly adjacent to the flange angles and greater than the shear midway between the rivet lines obtained under the previous assumption, and it probably corresponds more closely to the actual condition of the girder. The resulting pitch is $9190/2988 = 3.08$ inches, while that found by the other method is 2.99 inches, or about three percent less. If, however, in the other method the gross sections be used instead of the net sections, the resulting pitch is 3.03 inches, or only 1.6 percent less, and thereby indicates the magnitude of the error involved in the approximation made in deducing the formula in Art. 54.

The horizontal shear just below the flange angles is 84.5 percent of that at the neutral surface, while the average value of the horizontal shear for the entire section is 3483 pounds per linear inch, or 89 percent of that at the neutral surface. This value is also laid off in the diagram for comparison. Below the flange angles the abscissas represent not only the magnitudes of the horizontal shear per linear inch, but also the equal magnitudes of the vertical shear at the respective distances from the neutral surface. If it be assumed that the entire vertical shear is resisted by the web plate alone, the average vertical shear per linear inch is $294\,100/84 = 3501$ pounds, or 89.5 percent of the shear at the neutral surface. These values show how slight is the error involved in this assumption, and that if a proper allowance be made in adopting the safe unit

stress, the net area of the web section may be found with sufficient accuracy by assuming the shear to be uniformly distributed.

The length of the longitudinal splice plates for the web (Art. 56) may now be determined. The theoretic pitch of the flange rivets at the joint is over 5 inches, while the adopted pitch in this part of the span is 5 inches, and hence the pitch will be reduced near the joint to $2\frac{1}{2}$ inches. If the plates be extended to the left 29 inches, they will contain 11 rivets, $5\frac{1}{2}$ of which are required for the stress due to the web splice, and the remaining $5\frac{1}{2}$ for the increment of flange stress. Theoretically the plates need not extend an equal distance to the right of the joint, or toward the middle of the girder, but only far enough to contain 6 rivets, unless more are required to transmit their stress into the angles by single shear. If 6 rivets are used, they will develop the net strength of the plates. The total length of the plate will therefore be $29 + 16\frac{1}{2} = 45\frac{1}{2}$ inches.

In a plate girder where side plates or vertical flange plates are placed between the angles and the web, all the rivets through the side plates, whether passing also through the angles or not, may be counted in the number necessary to take the horizontal flange increment out of the web plate. To determine the number of rivets which must also pass through the vertical legs of the angles it must be considered that they shall provide sufficient strength in double shear to transmit that portion of the increment of flange stress which is to be taken by the angles and cover plates. The double shear will govern in this case, since its value will be less than their bearing either in both angles or in the combined web and side plates.

For a flange of the composition shown in Fig. 11 the riveting should be so designed as to transfer the increment of flange stress from the web to the several shapes composing the flange in the most direct manner.

A purely graphic method of determining the rivet pitch is described in a paper by E. SCHMITT in the Transactions of the American Society of Civil Engineers, vol. 45, page 550, June, 1901. In the discussion of this paper C. B. WING gives a series of diagrams for the solution of the same problem, covering a wide range of unit stresses and of the other factors involved.

The practical considerations which affect the spacing of the rivets both longitudinally and transversely are given in the next article.

ART. 60. LOCATION OF FLANGE RIVETS.

SPECIFICATION. — The pitch of rivets shall not be less than three diameters when on the same line, nor less than two and one-half times the diameter when staggered. The pitch in the direction of the stress shall never exceed six inches, nor sixteen times the thickness of the thinnest outside plate. When two or more thicknesses of plate are riveted together in compression members, the outer row of rivets shall not be more than four diameters from the side edge of the plate. No rivet-hole center shall be less than one and a half diameters from the edge of a plate, and, whenever practicable, this distance is to be increased to two diameters.

If the theoretic pitch at the end of the girder be less than that required by the specification, the thickness of the web must be increased accordingly. Sometimes three and one-half diameters is specified as the minimum pitch in one line. The smallest angle that will admit two rows of $\frac{7}{8}$-inch rivets is 5 inches (Art. 34), but the rivets in flange angles are usually placed in a single row whenever the vertical leg of the angle is less than 6 inches. While occasionally a single row is used in 6-inch angles, it is expressly forbidden in some specifications. In 8-inch angles it is customary to use two rows, but sometimes three rows are inserted. The location of the pitch lines or rows of rivets is given in Art. 34.

In order to facilitate shop work, the pitch of rivets in deck-plate girders is increased in regular groups from the minimum required at the ends to the maximum near the middle, the

number of changes being few. The object of limiting the maximum pitch according to the specification is to secure a close fit in construction. The pitch is made the same in both upper and lower flanges for economy in manufacture. No change in pitch should be made between two adjacent stiffeners unless necessitated by a flange splice. The spacing in the flange angles must be slightly modified at the location of the stiffeners so as to give sufficient clearance during construction. In through girders the pitch is uniform in each panel.

In the deck girder whose theoretic rivet pitch was determined in the last article the end pitch may be taken as 3 inches, since the pitch was found to be only slightly less in a distance of 2.7 feet from section 0′, or the center of the support, while a number of extra flange rivets may be placed in the foot or so which the girder extends beyond that section, and which will more than make up for the difference in pitch. As Fig. 70 shows that the maximum allowable pitch of 6 inches may only be extended to 7 feet on each side of the middle, it is preferable to omit that pitch and employ the 5-inch pitch for 15 feet from the middle. This arrangement will then leave only one intermediate pitch, that of 4 inches.

This specification permits two lines of rivets to be used to connect the cover plates to the angles, when the angles are 6 inches or less in width, but usually requires four lines of rivets when the cover plates are more than about 14 inches wide. If in order to keep down the number of plates it is necessary to widen them beyond the limits already indicated, it is best to increase the width sufficiently to allow a row of rivets on each side outside of the flange angles.

At the ends of cover plates the pitch should be reduced for a short distance so as to reduce the tendency to overstrain the rivets on account of the sudden change in flange section.

A sufficient number of rivets to transmit the full stress for which the plate is designed may be placed at the end of the plate with the smaller pitch. Sometimes this pitch is limited to the minimum used in the flange.

When only one row of rivets is used in each leg of the angles, the horizontal and vertical rivets should stagger, but when two rows of rivets are used in each leg of the angles, the adjacent rows in one angle, whether both are in one leg or not, should stagger. This arrangement places the rivets in the outer row of the horizontal leg of the angle opposite to those of the upper row of the vertical leg. Sometimes those in both rows of one leg are placed opposite points which are intermediate between the adjacent rivets of both rows in the other leg, but this is objectionable on the ground of reducing the net section throughout the span, except near the middle where the pitch is but slightly less, or equal to, the maximum allowed.

In the example given, it was found that the theoretic pitch for the rivets through the cover plates and angles is about 9 inches, which exceeds the maximum allowed. As these rivets must be spaced with regard to those through the vertical legs of the angles, the pitch must either be equal to theirs, or just twice as great, provided the resulting value does not exceed 6 inches. The values to be adopted can therefore be determined in accordance with these statements after the flange and web splices, ends of cover plates, and stiffeners are located on the drawing.

It should be added that a few specifications, including those of WADDELL, direct that the flanges of girders carrying the vertical load from the ties shall have their rivets spaced uniformly from end to end and at the minimum distance employed.

ART. 61. FLANGE SPLICES.

SPECIFICATION. — Splices in flange plates and angles must always be avoided when sufficiently long plates and angles are procurable, which will always be the case unless the span be abnormally long. Where flange splices are un-

avoidable, they must be so located that no two pieces of either the flange or the web shall be spliced within two feet of each other, and so that no flange and splice shall occur at any point where there is not an excess of sectional area above the theoretical requirements.

The saving in the cost of splices will usually compensate for the extra price which may be demanded for plates and angles of the greatest length obtainable. The object of distributing the splices in the different pieces, and of not allowing any flange splice to come too near a web splice, is to avoid abrupt changes in section which interfere with the proper distribution of stresses in the different members.

As the web splices occur at 12 feet from the middle of the girder, and the outer cover plate extends a few feet farther each way (see Fig. 69, Art. 58), the two angles will be spliced between the center of the girder and the two web splices respectively. The first cover plate will be spliced at two points, so that the second cover plate may be extended sufficiently to serve at each end as a splice plate.

Each of the $8'' \times 8'' \times \frac{3}{4}''$ angles has a net area of $11.44 - 1.50 = 9.94$ square inches, and is to be spliced by an $8'' \times 8''$ angle cut down so as to fit the face of the flange angle, and to have at least the same area. This requires the angle to be seven-eighths of an inch thick, and each leg cut down to about $7\frac{1}{8}$ inches. As the value of a $\frac{7}{8}$-inch rivet in single shear is 7220 pounds, 24 rivets are required to connect the splice angle on each side of the joint. With a pitch of $2\frac{1}{2}$ inches, which is the minimum allowed without reducing the net section of the flange (Art. 00), the length of the splice angle is a little over 5 feet. This would interfere with at least one pair of stiffeners, and hence it is desirable to reduce the length, which can be done by reducing the thickness of the splice angle to nine-sixteenths of an inch, and making up the sectional area by placing a $7'' \times \frac{9}{16}''$ flat on the vertical leg of the opposite flange angle, as shown in Fig. 72.

The flat requires 8 rivets at each end, and the angle 16 rivets, it being remembered that the bearing in the ¾-inch flange angle is greater than the double shear. Accordingly all the rivets in the vertical leg of the splice angle also pass through the flat, and its length is thereby reduced one-third. It should be noticed that these splices are located at points where there is an excess of flange area.

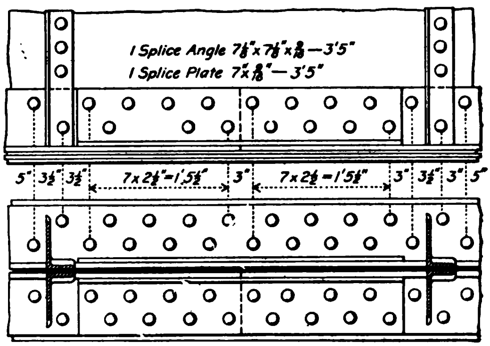


Fig. 72.

Let the nearer flange angle be spliced on the left, and the farther angle on the right of the middle of the girder, the same arrangement being also adopted for the upper flange, and all the splices located, so as not to interfere with any stiffeners.

The net section of the first cover plate is $7.88 - 0.88 = 7$ square inches (Art. 55), and as the second cover plate, which has the same section, is to be extended as a splice plate, single shear will govern the design of the connecting rivets. The number required on each side of the joint is $7 \times 17\,000/7220 = 17$ rivets; but as the rivets are arranged symmetrically in pairs, 18 rivets must be used.

Since the second cover plate is theoretically required at 17 feet from the end (Fig. 69, Art. 58), and the 4-inch theoretic pitch of flange rivets begins at 15 feet (Fig. 70, Art. 59), it is desirable to extend the 3-inch pitch over this splice. The second cover plate will then extend to about 12 feet 6 inches from the center of the support, making its entire length about 55 feet, while that of the middle portion of the first cover plate is 50.5

feet. These lengths are subject to a slight modification on account of the location of the stiffeners, which interfere somewhat with the regularity of the rivet pitch.

This extension of the 3-inch rivet pitch, and the use of a $2\frac{1}{2}$-inch pitch for about 2 feet at the end of the outer cover plate leaves less than 7 feet remaining for the 4-inch pitch. It should therefore be considered whether it may not be better to omit that pitch altogether.

WADDELL'S specification requires that every non-continuous flange piece shall be fully spliced so that the splicing plates and rivets shall have a calculated strength at least 25 percent greater than that of the net section spliced. Under this requirement the computation given above would have to be changed accordingly.

ART. 62. LATERAL BRACING.

SPECIFICATION. — The lateral bracing shall be proportioned for a static wind load of 150 pounds per linear foot on each system. The system connected to the loaded flanges shall be proportioned also for a moving wind load of 300 pounds per linear foot. The compression flanges of the girder shall be so stiffened laterally that the unsupported length shall not exceed 12 times the width of flange. All members shall be so proportioned that the tensile unit stress shall not exceed 17 000 pounds per square inch, nor the compressive unit stress to exceed 17 000 pounds per square inch reduced in proportion to the ratio of the length to the least radius of gyration of the section, by the following formula: $p = 17\,000/\left(1 + \frac{1}{11\,000} \cdot \frac{l^2}{r^2}\right)$, in which p is the permissible working stress per square inch in compression, l the length of piece in inches between centers of connections, and r the least radius of gyration of the section in inches. No compression member in the wind bracing shall have a length exceeding 120 times its least radius of gyration. For members of any importance, more than two rivets are to be used for each connection. For unit stresses on rivets, see Art. 56. In field riveting the number of rivets found by the specified unit stresses shall be increased 25 percent if driven by hand, or 10 percent if satisfactory power riveters are used.

In the specifications which refer to the spacing of cross-frames, its value ranges from 12 to 20 feet. In order to make them equidistant in an 80-foot span, it is necessary either to space them 20 feet, 16 feet, or 13 feet 4 inches. Adopting the intermediate distance the panel length of the upper lateral system becomes also 16 feet for a Warren type of bracing, and may be reduced to 8 feet by adding substruts at every panel point. Every alternate substrut is a member of the cross-frame or transverse bracing. The upper lateral system holds in line the compression flange of each girder. As the cover plates are 18 inches wide, the allowable unsupported length is 18 feet. The substruts which form no part of a cross-frame are therefore not required on this account, but may be inserted in accordance with the best practice. They will be omitted in the lower system. The skeleton diagrams of the upper and lower lateral systems are shown in Figs. 73 and 74 respectively.

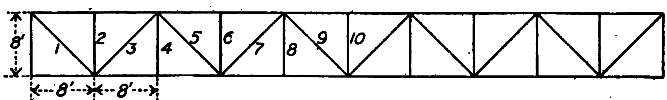


Fig. 73.

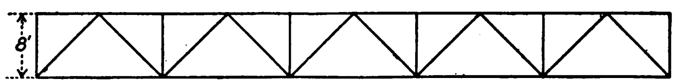


Fig. 74.

For the specified wind loads the maximum stresses in the diagonals are as follows: $S_1 = \pm 22\,900$, $S_3 = \pm 18\,200$, $S_5 = \pm 13\,800$, $S_7 = \pm 9700$, and $S_9 = \pm 5900$ pounds. As the double signs are due to the wind blowing in opposite directions, and the reversals of stress do not take place in rapid succession as in stresses due to live load, it is customary not to design the lateral system for alternate stresses.

Since the ratio of the length l of a diagonal to its least radius of gyration r is to be limited to 120, the least allowable radius

of gyration is $106/120 = 0.88$ inch, and a reference to one of the handbooks shows that for a single angle no smaller size than $6'' \times 4''$ or $5'' \times 5''$ can be used. For the former size, in which l/r is 120, the specified column formula gives an average compression per square inch of 7360 pounds, and hence the required area of the end diagonal is $22\,900/7360 = 3.11$ square inches. The area of a $6'' \times 4'' \times \frac{3}{8}''$ angle is found to be 3.61 square inches, while r is 0.88 inch, the value assumed. The thickness of this angle is the least allowed, but on account of the eccentric end connections of the angle its sectional area will probably have little to spare.

Let an investigation be made to see whether the sum of the stresses in the outer fiber, due to both the column action and the eccentric connection, falls within the allowable limit of 17 000 pounds per square inch. First, let the angles be riveted to the connecting plates by the 6-inch leg, and let bending in a vertical plane be considered. According to the handbook, the distance from its center of gravity to the back of the longer flange is 0.94 inch, its moment of inertia I about the neutral axis parallel to the longer flange is 4.90 inches^4, and the corresponding value of the radius of gyration r is 1.17 inches. The angle tends to bend so that the concave side is on the back of the longer flange, and since $l/r = 106/1.17 = 91$, the maximum compressive stress on that side is

$$S' = \frac{22\,900}{3.61}\left(1 + \frac{91 \times 91}{11\,000}\right) = 11\,100 \text{ pounds per square inch.}$$

The bending moment due to the eccentric connection is $22\,900 \times 0.94 = 21\,500$ pound-inches, and by means of the formula deduced in Mechanics of Materials, Art. 117, the compressive stress in the outer fiber on the same side of the angle which is due to this moment is

$$S'' = \frac{21\,500 \times 0.94}{4.90 - \dfrac{22\,900 \times 106 \times 106}{9.6 \times 29\,000\,000}} = 5100 \text{ pounds per square inch};$$

in which 29 000 000 is the coefficient of elasticity. The total stress is therefore 11 100 + 5100 = 16 200 pounds per square inch.

A similar computation for the second case, when the angles are connected by the 4-inch leg, gives 8800 + 6900 = 15 700 pounds per square inch. If, in the first case, the bending moment due to its own weight be included, the unit stress is increased 200 pounds per square inch. Repeating the computation for an angle $5'' \times 5'' \times \frac{3}{8}''$, the result is 9000 + 5700 = 14 700 pounds per square inch. The area of both angles is the same, and a comparison of the maximum unit stresses indicates their relative strength.

By connecting the other leg of the angle to the connecting plate by means of an angle clip, the eccentricity which tends to produce bending in a horizontal plane may be eliminated, but that for bending in a vertical plane still remains, since the entire stress is transmitted by the rivets through the connecting plate. Since the eccentricity is very small in the plane in which the angle tends to bend as a column, — that is, in a plane perpendicular to the neutral axis, with respect to which the radius of gyration is a minimum, — this investigation does not need to be carried further. It may be noted that the net section of one leg of the angle is sufficient to transmit the tension in the end diagonal, as this requirement is sometimes specified when only one leg of the angle is attached. Although the stresses in the remaining diagonals are less than in those at the end of the span, the same size is required throughout, since l/r is limited to 120.

The length of the lateral braces which are perpendicular to the girders is 82 inches, and hence the radius of gyration must

not be less than $82/120 = 0.68$ inch. As the stress is only 1800 pounds, a $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ angle will have abundant strength. No narrower angle than $3\frac{1}{2}$ inches will admit a $\frac{7}{8}$-inch rivet, according to the standard given in Art. 34. The conditions already referred to require the same sized laterals to be used in the lower system as in the upper one, although the stresses are less than one-third as large.

Since the connecting rivets are in single shear, their shearing value will govern, and as the rivets through the laterals are field and not shop rivets, their number must be increased 25 percent for hand riveting. These conditions require 4 rivets in the connection of the end diagonal, but an additional rivet is needed on account of its eccentricity. This result may be tested as follows: The longitudinal shear in each rivet is $22\,900/5 = 4580$ pounds. If three rivets be placed in the pitch line which is $1\frac{3}{4}$ inches from the back of the 5-inch angle and two rivets in the line which is 2 inches from the other one, the center of gravity of the shearing surfaces of the rivets is $1.75 + 0.8 = 2.55$ inches from the back of the angle, or 1.16 inches farther from it than the center of gravity of the angle. The moment of rotation in the plane of the shearing surfaces, caused by the stress in the angle, is therefore $22\,900 \times 1.16 = 26\,570$ pound-inches, and this moment produces a shear in each rivet whose value is directly proportional to its lever arm. Two of the rivets are 5.1, two are 2.8, and one is 0.8 inches from the center of rotation, and if the shear in the most distant rivet is P, the moment of the shear in all the rivets is

$$(\overline{5.1}^2 + \overline{5.1}^2 + \overline{2.8}^2 + \overline{2.8}^2 + \overline{0.8}^2)\ P/5.1 \text{ pound-inches.}$$

Equating this to the moment of rotation and solving for P, its value is found to be 1980 pounds. The direction of this shear is perpendicular to the lever arm of 5.1 inches, while that of the shear of 4580 pounds is parallel to the axis of the angle. Their

resultant is found graphically to be 5280 pounds. The allowable stress for field rivets is 20 percent less than for shop rivets, or 5770 pounds. Five rivets are therefore required. The moment of rotation in a vertical plane tends to produce tension in some of the rivets, but as the connecting plate bends easily in that direction the rivet tension must be small. No less than three rivets should be used in connecting any lateral, even though that number is not theoretically needed.

Although the laterals are usually designed to take the wind stress only, it should be remembered that their principal duty is to resist the lateral vibrations caused by the live load passing over the bridge at full speed. In view of the great increase in live loads, it is a question whether the assumption that these stresses do not exceed those computed for the wind pressure leads to a sufficient provision for lateral stiffness. When it is considered that these vibrations cause rapid reversals of stress, a material increase in lateral stiffness would be secured by treating the wind stresses as live-load stresses, and designing the laterals for alternate stresses.

In designing members for alternate or reversed stresses, one of the best specifications is to find separately the areas required for both tension and compression, and to add three-fourths of the smaller area to the larger one in order to obtain the total sectional area of the member. The rivets, however, are to be computed for the sum of the two stresses. On applying this to the lateral system under consideration, the end diagonal of the upper system must be increased to $\frac{1}{2}$ inch in thickness, the rest remaining unchanged. If clips be used so as to reduce the eccentricity of the connections, the numbers of field rivets required in the diagonals of the upper system are respectively 8, 7, 5, 4, and 3.

In some cases it may be advisable to go a step further and increase the stresses by allowance for impact. Practically the

same result may, however, be secured by increasing the prescribed wind pressure. The Atchison, Topeka, and Santa Fé Railway uses a moving wind load of 500 pounds per linear foot and an equal static wind load in its standard designs for plate-girder bridges.

ART. 63. TRANSVERSE BRACING.

The form and composition of the transverse bracing were described in Art. 43. The object of the intermediate cross-frames is to increase the general stiffness of the bridge, and all the angles composing its horizontal and diagonal braces will be taken as $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ for the reasons indicated in the previous article. Each connection should have three rivets, and where the diagonals cross they should be riveted to a small connecting plate.

The end cross-frame must transfer the reaction of the upper lateral system to the support. This reaction is 18 000 pounds. If it be assumed that one-half of the reaction is transferred to the support by each diagonal, the areas required in all the members will be less than those of the small angles already adopted for the intermediate cross-frames. As it is very important that the end bracing shall be rigid, it is best to use the larger shapes which are employed in the best practice. Each diagonal may be composed of one angle $5'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$, the upper horizontal of two angles $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$, and the lower horizontal of two angles $5'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$. (See also Plate I, Art. 69, and the references in Art. 49.)

ART. 64. BEARINGS AT SUPPORTS.

The principal types of expansion bearings now in use were described in Art. 44. As indicated in that article there are considerable differences in the specifications with regard to the length of span below which sliding bearings and above which

roller bearings are to be used respectively. Most specifications give this span as 75 or 80 feet. A few require rollers when the span exceeds 60 feet, while others permit sliding up to 85 or 90 feet. Several specifications require some form of rocker bearing or hinge joint for spans from 50 feet to 65 or 70 feet, and in exceptional cases even beyond this limit.

Hinge bolsters should be used in combination with rollers in order to secure a uniform distribution of the load on the rollers with whatever deflection the girder may sustain under its live load.

The expansion end of the girder shall be free to move longitudinally for a variation in temperature of 150 degrees Fahrenheit, but must be anchored against lifting or moving sideways.

When cast-steel shoes are employed, as in Figs. 51 and 52, their design must make the following provisions: Adequate bearing area of the vertical ribs on the pin; a pin of sufficient diameter to resist the bending moment produced on account of the outer ribs of one shoe being farther apart than those of the other shoe; vertical longitudinal ribs of the necessary strength as double cantilevers to carry the load from the stiffeners at the ends of the upper shoe to the pin, and in the lower shoe to distribute the pin reaction as a uniform load to the rollers; and vertical transverse ribs and bearing plates of ample thickness to make the distribution of pressure uniform transversely. These ribs have the additional duty of stiffening the longitudinal ribs and aiding them to resist any transverse horizontal thrust that may be brought upon them in service. If the greatest allowable pressure in pounds per linear inch is specified as $p = 600\,d$, in which d is the diameter of the rollers, their aggregate length is found on dividing the gross reaction of the girder including the shoes by this allowable bearing. The bearing area of the bed plate under the rails must be such as not to exceed a safe

value for the material composing the bridge seat. Where impact is taken into account, this value may be taken as about 400 pounds per square inch. WADDELL specifies permissible pressures for ten different materials.

The anchor bolts at the fixed end must be designed to take the combined shear and tension due to the tendency for the cast-iron bolsters to slide and overturn when the brakes are applied to the train crossing the bridge at full speed. The horizontal tractive load thus applied to the girders is to be taken as 20 percent of the greatest live load that can be placed on the bridge.

When bolsters are built up of plates and shapes, the same general method of design is followed. Whether two or three vertical plates shall be used depends upon the size of the girder. Transverse webs should be employed so as to secure the proper distribution of loading in that direction, unless this can readily be done by bearing plates of moderate thickness without too great a stress in flexure. The vertical legs of the connecting angles should be wide enough to allow two rows of rivets. It is often specified that no bearing plate, bed plate, vertical plate, or connecting angle should be less than three-quarters of an inch in thickness, and sometimes the minimum for the bed plate is made seven-eighths of an inch. No rollers less than 3 inches in diameter are allowed, while the best practice makes the minimum diameter 4 inches.

The segmental rollers with parallel sides shown in Fig. 48, Art. 44, are the standard adopted by the bridge department of the New York Central and Hudson River Railroad.

For additional information relating to the design of segmental rollers see Art. 81. References to the bearing power of friction rollers are given in Art. 98. Formulas for the investigation and design of cylindrical rollers are deduced in Mechanics of Materials, Art. 107.

ART. 65. ESTIMATE OF WEIGHT.

The following weights are computed with the aid of the tables in a handbook:

MATERIAL FOR ONE-HALF OF THE GIRDER.

Flanges:		
4 angles, 8″ × 8″ × ¾″ × 40′ 10″, @ 38.9 lbs.	6354 pounds.	
2 cover plates, 18″ × $\frac{7}{16}$″ × 40′ 10″, } @ 26.79 lbs.		
2 cover plates, 18″ × $\frac{7}{16}$″ × 27′ 6″, } @ 26.79 lbs.	4532	10 866
2 cover plates, 18″ × $\frac{7}{16}$″ × 16′ 3″, } @ 26.79 lbs.		
Flange splices:		
2 cover angles, 7⅛″ × 7⅛″ × $\frac{9}{16}$″ × 3′ 5″, @ 26.6 lbs.	182	
2 plates, 7″ × $\frac{9}{16}$″ × 3′ 5″, @ 13.39 lbs.	91	273
Web:		
1 plate, 84″ × $\frac{7}{16}$″ × 28′ 9¾″, } @ 124.96 lbs.		
½ plate, 84″ × $\frac{7}{16}$″ × 23′ 11¾″, } @ 124.96 lbs.	5099	5 099
Web splice:		
2 plates, 13″ × ⅜″ × 5′ 8″, @ 16.58 lbs.	188	
4 plates, 7″ × ⅜″ × 3′ 8½″, @ 8.93 lbs.	132	320
Stiffeners:		
24 angles, 6″ × 3½″ × ⅜″ × 6′ 10½″, @ 11.7 lbs.	1931	
1 angle, 7″ × 3½″ × $\frac{7}{16}$″ × 6′ 10½″, @ 15 lbs.	103	
6½ fillers, 3½″ × ¾″ × 5′ 8″, @ 8.93 lbs.	329	
2 plates, 20″ × ¾″ × 5′ 8″, @ 51 lbs.	578	2 941
1 end cover plate, 18″ × ⅜″ × 7′ 4″, @ 22.96 lbs.	168	168
Total		19 687

ONE-HALF OF UPPER LATERAL SYSTEM.

Braces: 1 angle, 5″ × 5″ × ½″ × 9′ 4″, @ 16.2 lbs.	151
4 angles, 5″ × 5″ × ⅜″ × 9′ 4″, @ 12.3 lbs.	459
2½ angles, 3½″ × 3½″ × ⅜″ × 6′ 10″, @ 8.5 lbs.	145
10 connecting angles, 5″ × 3½″ × ⅜″, aggregating 6′ 2″, @ 10.4 lbs.	64
16 connecting ⅜″ plates, aggregating 22.56 sq. ft., @ 15.3 lbs.	345
	1 164

ONE-HALF OF LOWER LATERAL SYSTEM.

Braces: 5 angles, 5″ × 5″ × ⅜″ × 9′ 4″, @ 12.3 lbs.	674
4 connecting angles, 5″ × 3½″ × ⅜″, aggregating 2′ 4″, @ 10.4 lbs.	24
8½ connecting ⅜″ plates, aggregating 13.72 sq. ft., @ 15.3 lbs.	210
	808

END CROSS-FRAME.

2 angles, 5″ × 3½″ × ½″ × 8′ 3″, @ 13.6 lbs.	224
2 angles, 3½″ × 3½″ × ⅜″ × 6′ 7½″, @ 8.5 lbs.	113
2 angles, 5″ × 3½″ × ⅜″ × 6′ 7½″, @ 10.4 lbs.	138
4 connecting angles, 7″ × 3½″ × $\frac{7}{16}$″ × 1′ 8¾″, @ 15 lbs.	105
5 connecting ⅜″ plates, aggregating 9 sq. ft., at 15.3 lbs.	138
6 ⅜″ washers	5
	423

INTERMEDIATE CROSS-FRAME.

2 angles, 3½″ × 3½″ × ⅜″ × 8′ 10″, @ 8.5 lbs.	150
2 angles, 3½″ × 3½″ × ⅜″ × 6′ 9″, @ 8.5 lbs.	115
5 connecting ⅜″ plates, aggregating 4.7 sq. ft., @ 15.3 lbs.	72
	337

DEAD LOAD FOR ONE GIRDER, EXCLUDING TRACK.

1 girder, 2 × 19 486 lbs.	39 374 pounds.
½ of upper lateral system	1 164
½ of lower lateral system	808
1 end cross-frame	723
2 intermediate cross-frames	674
3036 pairs of rivet heads, @ 0.452 lb.	1 372
Gross weight for a length of 81′ 8″	44 115 pounds.
Net weight for a length of 80′ (the span)	43 215 pounds.

The net weight was assumed to be 45 200 pounds, and the difference of 1985 pounds is found to be less than one percent of the sum of the equivalent live load and the actual dead load. The stresses will therefore not require revision.

The following table gives the weights of the various parts of the structure, exclusive of track and pedestals, and the corresponding percentages of the entire weight:

	Weight in Pounds.	Percentage of Total Weight.	
Flanges	21 772	49.4	50.6
Flange splices	546	1.2	
Web	10 198	23.1	38.7
Web splices	640	1.5	
Stiffeners and end cover plates	6 218	14.1	
Half upper lateral system	1 164	2.6	7.6
Half lower lateral system	808	1.8	
Cross-frames	1 397	3.2	
Rivets	1 372	3.1	3.1
Total	44 115	100.0	100.0

As the weight of the girder depends not only upon the given loads, but also on the unit stresses and many other details prescribed by the specifications, it is difficult to deduce a general formula for the weight. The above analysis, however, makes it possible to estimate the total weight very closely at an early stage of the design, for the combined weight of the flanges and web plate is 73.2 percent of the entire weight of the girder and bracing. This percentage has but a small range for different specifications in combination with a large range of live load.

At first, let the weight per linear foot be assumed as six to seven times the span in feet, the larger value being used for the heaviest live loads, and then let the web section and the composition of the flange be designed in accordance with the specifications adopted. The approximate lengths of the cover plates may be quickly found by dividing the maximum ordinate of the moment diagram in proportion to the respective flange areas, and locating the corresponding ordinates. If the weight of these items be increased by about one-third, the result will differ but little from the final estimate of the weight.

ART. 66. ECONOMIC DEPTH.

It is of interest to observe what absolute as well as relative variations in the weight will be obtained by changing the depth of the girder. The weight of the flanges varies inversely as the effective depth, while that of the web, together with its splices and stiffeners, varies nearly as the depth of the web plate for relatively small changes in depth, and these two depths differ only by amounts ranging from 1.35 inches at the center to 2.1 inches at the end of the span in this example. Slight changes from the economic depth do not appreciably affect the weight of the girder, hence these variations in depth should produce about equal changes in the weights of the flanges and of the web with their respective details. The minimum material results when these weights are about equal, as was shown in Art. 11.

Experience shows that in order to compute the corresponding weights for different depths the results will usually be sufficiently close by assuming the weights of the flanges to vary inversely as the depths of the web plate. For a depth of 96 inches the weight of the flanges and their splices will be about $(21\,772 + 546)84/96 = 19\,530$ pounds, while the weight of the web and its details will be about $(10\,198 + 640 + 6218)96/84 = 19\,490$ pounds. This shows that 96 inches, or one-tenth of the span, is the depth which requires the minimum material. The reduction in the total weight is only about 350 pounds, and the percentages are 44.6 and 44.6 instead of 50.6 and 38.7 given in the table in the preceding article. Considerations relating to the cost of manufacture and the limitations imposed by required clearances and the grade line of the railroad, generally make the true economic depth somewhat less than that which gives the minimum weight.

ART. 67. CAMBER.

The extensive adoption of plate girders for increasing spans in recent years has led to the practice of providing a camber so

that when the bridge is loaded the track will not sink below the horizontal. According to the standards of the Northern Pacific Railway, for a span of about 100 feet the web plates are spliced so as to give a camber of one inch before the girders are erected, and afterwards the cross-ties are notched so as to leave a camber of $\frac{1}{2}$ inch in the finished unloaded bridge. For a span of 80 feet these amounts are reduced 25 percent.

In the track elevation in Chicago, camber was provided in some girders of still shorter spans, five-eighths of an inch being put into girders of 68 feet span, one-half of which remained in the unloaded spans when completed. In some others of the same span the initial camber was $1\frac{1}{4}$ inches and the final camber $\frac{1}{2}$ inch. Some specifications, however, prescribe that plate girders shall have no camber.

ART. 68. DETAIL DRAWINGS.

Instead of publishing the general plans of the girder whose design, with the exception of a few minor features, is given in this chapter, there are shown on Plates I and II in the next article the full detail drawings of a girder bridge of the same span, this being one of the standard plans of the Northern Pacific Railroad. These plans are practically shop drawings, as the full dimensions are given for every piece and all the members and rivets are located. The notes on Plate I give the loads for which the girders were designed.

The student should examine Plate I carefully and note the differences between the details shown and those designed in this chapter. Special attention is called to the wooden floor and its connections; to the composition of the flanges; to the positions of the web and flange splices; to the modification of the rivet spacing in the flanges on account of the stiffeners and splices; to the intersection of the rivet lines of the lateral

angles in the web plate of each girder; to the end rivet in nearly every lateral being a flange rivet; and to the connection by both legs of the horizontal transverse braces to the girders.

The details of the end bearings, whose complete shop drawings are given on Plate II, require no additional explanation. The care with which every detail has been designed is manifest. The form of the segmental rollers and of the roller plate is the Morison standard, which is described in Art. 81.

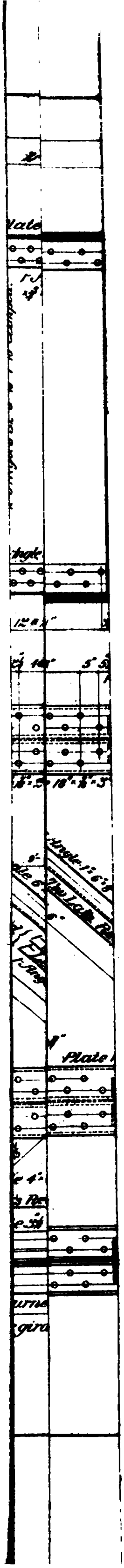
The student will also find it advantageous to make a comparative study of the details of the plate-girder bridges, to which references are given in Art. 49.

ART. 69. STANDARD PLANS.

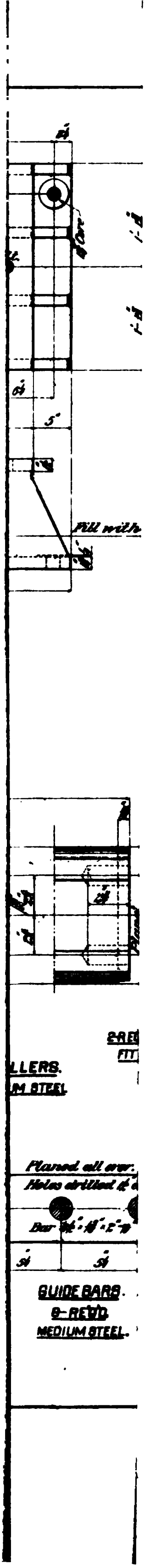
The modern movement in American practice to standardize the details of construction has been extended by some railroads to the design of plate-girder bridges. Among these may be mentioned the Northern Pacific, the Atchison, Topeka and Santa Fé, and the Great Northern railroads.

The standard plans of the Northern Pacific Railway include complete drawings of deck plate-girder bridges from 25 to 100 feet in span, and of through bridges from 30 to 100 feet, both kinds varying by 5 feet in span. One of these plans is shown on Plate I. In the Journal of the Western Society of Engineers, vol. 6, page 51, Feb., 1901, may be found a paper on Northern Pacific Standard Bridge Plans by RALPH MODJESKI, who prepared them as consulting engineer. The plates accompanying the paper include the plans of both deck and through plate-girder bridges for spans of 60 and 100 feet respectively. A diagram of weights is also given.

On the Atchison, Topeka and Santa Fé Railway the standard plate-girder bridges are divided into four classes. Class A includes deck girders from 26 to 105½ feet in length, out to out,



Plate



Fill with
LLERS.
Planed all over.
GUIDE BARS.
8-REQ'D.
MEDIUM STEEL.

designed for economy of weight. The depth ranges from a little over one-seventh to a little less than one-tenth of the effective span. The cross-frames are only from 8 to 10 feet apart and divide the lateral systems into panels, intersecting diagonals being employed in each panel of both systems.

Class B consists of deck girders from 26 to 85 feet in length, which are intended for locations where it is necessary to make them as shallow as the limits of deflection permit. For lengths from about 45 to 85 feet the general arrangement is about the same as for class A, except that the depth is reduced to one-thirteenth or one-fourteenth of the span. The flanges have vertical side plates to avoid too many cover plates and to accommodate the larger number of rivets needed to connect the flanges to the web plates. For lengths from 26 to 42 feet, four lines of girders are used, and they are so spaced that each track rail is midway between a pair of girders. The depth varies from 2′3″ to 2′8½″, which ranges from one-eleventh to one-fifteenth of the effective span. There is no lateral system in this case, but the four girders are connected by cross-frames with solid webs at intervals ranging from about 6 to 11½ feet. These bridges are proved by experience to have unusual lateral stiffness.

Class C includes through girders from 60 to 105½ feet in length, designed with long panels so as to economize material in the floor system. The girders have the same depths as in class A. The panels vary in length from about 14½ to 17 feet, and the floor beams are about 3½ feet deep. In the long spans four lines of stringers are used in order to reduce the economic depth of the floor. The spacing of the girders on tangents is 17 feet 2 inches.

Class D consists of through girders from 26 to 105½ feet in length, in which the floor system is designed as shallow as possible without reference to economy in weight. The panel lengths

vary from about 8 to 12 feet, and four lines of stringers are used for all spans.

Class B is not employed for lengths exceeding 75 feet, as the saving in depth would not warrant it, class C or D being substituted for it under these conditions. Classes A, B, and D have lengths increasing by increments of 3 to 5 feet, and in classes C and D additional plans are made adapted to curves of 5 and 10 degrees. The weights of the bridges increase in the order of the class letters for any given span, the shipping weights for a span of 60 feet, for example, comparing as the percentages 100, 121, 156, and 176. No expansion rollers are used in any case, but rockers are employed at one end in spans exceeding 75 feet.

CHAPTER VIII.

DETAILS OF RAILROAD PIN BRIDGES.

ART. 70. FORMS OF TRUSSES.

A comparison of the leading bridge specifications and railroad standards indicates that the preferred lower limit of span for plate girders ranges from 15 to 26 feet, that for riveted trusses from 75 to 100 feet, and that for pin-connected trusses from 120 to 150 feet. The New York Central and Hudson River Railroad, however, does not use pin-connected trusses for spans less than about 200 feet.

The riveted trusses are most frequently made either of the Warren type or of the Warren with sub-verticals, the Pratt truss being employed to some extent for the longer spans. The New York Central and Hudson River Railroad introduced in 1899 riveted trusses of the Baltimore type for spans from 100 to 200 feet, which prior to that time had been applied only to pin-connected trusses and to spans exceeding the larger limit named. Some details of riveted trusses are given in Chapter XI.

The Pratt is the prevailing type for the shorter spans of steel pin-connected trusses. The Warren truss with sub-verticals has been used in a few cases like that on the terminal improvements at Providence, R. I. (see Railroad Gazette, vol. 27, page 457, July 12, 1899), and that on the terminal improvements at Richmond, Va. (see Engineering News, vol. 44, page 379, Nov. 29, 1900). Formerly Warren pin trusses were employed more frequently, but it appeared later as though they would go out

of use entirely. Pegram trusses are used to a very limited extent on the Union Pacific and several other western railroads.

As the spans increase, the Pratt trusses are modified by curving the upper chord, and for still larger spans the panels are subdivided as in the Baltimore and the Pettit trusses. WADDELL's specifications indicate the preference of Pettit trusses for all spans above 250 feet, but a few have been built of slightly shorter span. While most of the newer simple truss bridges exceeding 300 feet in span are of the Pettit type, the Baltimore has been used up to 440 feet, as in the case of the Bellefontaine bridge erected in 1893. (See Fig. 105, Art. 80.)

ART. 71. OPEN FLOOR AND STRINGERS.

In through bridges there are generally two stringers to a track spaced from $6\frac{1}{2}$ to 8 feet apart, which support the track ties. The details of the ties, guard rails, etc., are about the same as for deck plate-girder bridges, except that alternate ties are frequently extended the full width for a footwalk. A few railroads, like the Boston and Maine, use four lines of stringers under each track, the main stringers being placed directly under the track rails, while the safety stringers are about $2\frac{1}{2}$ feet outside of the others. The continuity of the spacing of the cross-ties is broken by the floor beams, which support the stringers; but as the top flange of the floor beams is seldom more than a few inches below the tops of the ties, a derailed wheel will pass over the wider space in safety.

In some deck bridges of short span the ties are extended over the full width of the bridge and rest upon the chords of the trusses, as in the case of deck plate girders. As the span increases and with it the spacing of the trusses, this type of floor increases in cost and deflection, and is replaced by one of the same kind as that used for through bridges. In this case

the upper chords of the trusses frequently act also as safety stringers. See the report on bridge floors, to which reference was made in Art. 45.

When the panels are very short, the stringers may consist of I-beams, but generally their construction is similar to that of plate girders of short span. The flanges either consist of two angles or of two angles with one cover plate. The practice of not allowing cover plates is becoming quite prevalent, since it affords a better bearing for the ties, and simplifies the work of track maintenance. In some cases the web is extended $\frac{1}{2}$ or $\frac{3}{4}$ inch above the flange angles, thus obviating the necessity of notching the ties for the full width of the flange.

The stringers of each track are united by a lateral system of the Warren type attached to the upper flanges and by an intermediate cross-frame. Both of these features are used in long panels, and only one of them in short panels, some engineers using the lateral system in this case, while others use the cross-frame only. A cross-frame is also inserted at the ends of end stringers when there is no floor beam at the end of the bridge. The elevation of an intermediate stringer and of part of an end stringer, together with that of a cross or sway frame, is shown on Plate III. It will be noticed that there are no intermediate stiffeners in this example.

While the lateral system of stringers is generally of the simple Warren type, sub-struts are occasionally employed at the other panel points, as well as where the cross-frames are placed. On the inset of Engineering News, Jan. 11, 1900, may be seen an example where a double intersection Warren bracing is used. This arrangement, however, is quite unusual.

In through bridges the ends of the stringers are usually riveted to the webs of the floor beams between their flange angles by means of pairs of connecting angles and of bracket

angles, as indicated on Plates III and IV, Art. 82. Sometimes, however, the upper flange angles are extended over the floor beam, and the web cut out so as to clear the flange of the floor beam. (See Fig. 75.) Splice plates connect the tops of the adjacent stringer flanges, thus making them practically continuous and relieving the upper rivets in the connecting angles from tension when the adjoining panels are loaded. This arrangement also permits the ties to be spaced uniformly.

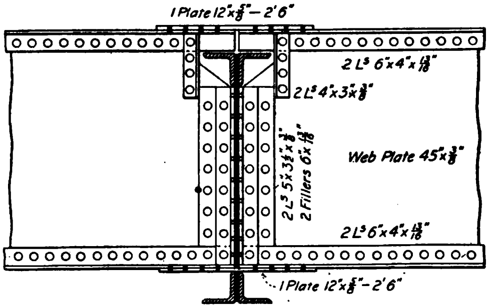


Fig. 75.

In deck bridges the stringers frequently rest on top of the floor beams, as illustrated on Plate V. The lateral system of the bridge may then be connected to the bottom of the stringers, the top of the floor beams, and the bottom of the chords of the trusses, and thereby avoid bending the floor beam horizontally by the tractive force developed on applying the brakes to the train.

When, however, the stringers are connected to the webs of the floor beams and the lateral system is connected to the top flanges of both floor beams and stringers, the web of the stringer may be extended far enough above the regular flange so as to attach secondary flange angles, on which to receive the ties. The projecting web and secondary flange are cut to allow the laterals to pass. This arrangement was adopted in the New Glasgow bridge, whose characteristic details are shown in Engineering Record, vol. 43, page 241, March 16, 1901.

The longest stringers in this country are those of the Delaware river bridge on the Pennsylvania Railroad, their span being 33 feet $3\frac{3}{4}$ inches.

ART. 72. SOLID FLOORS.

Several types of the trough floors described in Art. 46 are used in pin-connected truss bridges as well as in girder bridges. Some of the references given in Art. 47 contain descriptions and illustrations of their details when so applied. In an article on the Willamette bridge at Portland, Ore., in Railroad Gazette, vol. 21, page 260, April 19, 1889, the drawings show a splayed-channel trough floor riveted to the sides of the stiff lower chord of the trusses. In Engineering News, vol. 36, page 406, Dec. 17, 1896, may be seen the application of a trough system like Fig. 64, Art. 46, to the floor under the double-track railroad of the double-deck highway and railroad bridge at Rock Island, Ill. The floor is laid upon four lines of stringers, and continuous plates, $20'' \times \frac{3}{8}''$, are placed under the rails and riveted to the troughs so as to form an effective lateral bracing.

In the 348-foot span of the Victoria bridge at Montreal, the double tracks are laid on a continuous half-inch floor plate which is supported by transverse 24-inch I-beams spaced only about 14 inches apart. These I-beams are connected to the webs of longitudinal plate girders lying in the planes of the trusses and riveted to the posts below the lower chords. Longitudinal plates, $10'' \times \frac{1}{2}''$, are riveted on top of the floor plates under each rail.

The inset of Engineering News, Aug. 24, 1899, shows the plan of a solid floor built up of 12-inch channels and plates on the upper deck of the Wells Street bridge in Chicago. Two channels with their webs vertical, their flanges toward each other, and their backs $11\frac{3}{4}$ inches apart are connected by a top flange plate. Similar pairs of channels and cover plates are spaced 12 inches apart in the clear and connected by 12-inch channels with their webs horizontal and their backs at about the

mid-height of the vertical channels. The ties for the tracks of the Northwestern Elevated Railroad were laid directly in the shallow troughs of this floor without any ballast.

ART. 73. FLOOR BEAMS.

The floor beams of truss bridges are similar to those of through plate-girder bridges. The objection to cover plates in the case of stringers does not hold for floor beams. In through trusses a part of the web has to be cut away in order to clear the lower chord, and in order to secure enough space for rivets in the end

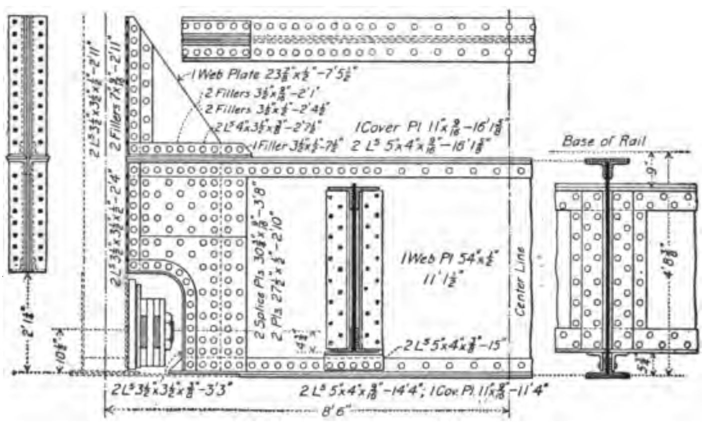


Fig. 76.

connecting angles the web must be extended beyond the upper flange. One form of construction is shown in Fig. 76. The web is spliced so that the end plate may extend up the required distance. The splice plates are continued to the end so as to act also as filler plates and to aid in strengthening the web around the cut. The connecting angles pass over the vertical legs of the upper flange angles, neither of the legs being cut away, and the curved angles pass over the lower end of the connecting angles. An additional pair of filler plates is there-

fore required under the curved angles, and they are extended beyond the angle to give increased strength and to simplify the construction. The lower flange and the bottom of the post are connected by a plate to which the diagonals of the lateral system are also attached.

In Fig. 77 is shown the end of a floor beam in the Pratt truss whose side elevation is given in Fig. 111. The inner edge of the extended web plate is stiffened by a pair of small angles. Several other special features will be noticed, especially the stiffeners between the stringer connections. Another form for a through bridge is shown on Plate III. Sometimes the lower flange angles are bent up outside of the stringers to take the place of the separate inclined angles, in which case another pair of short horizontal angles is riveted to the bottom of the web plate as shown on Plate IV. See also Engineering Record, vol. 43, page 244, March 16, 1901. When the floor beam is not extended down past the lower chord, the eccentric connections of the lateral system cause a bending moment in the bottom of the post which is avoided in the forms just described.

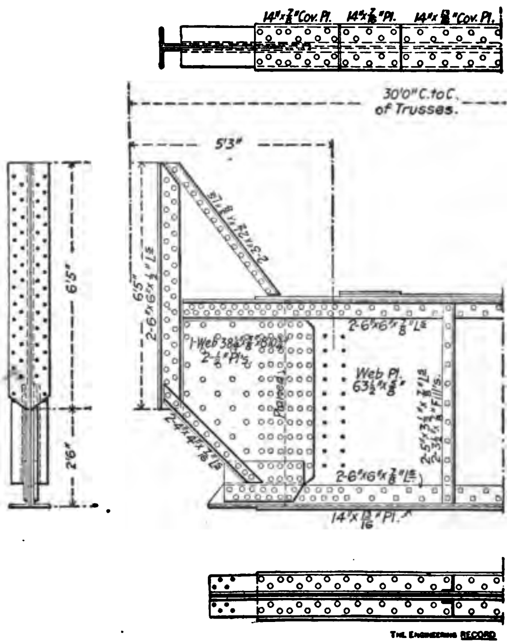


Fig 77.

In the Port Perry bridge over the Monongahela river this result is secured in another way. A trapezoidal web plate stiffened with angles is riveted to the bottom of the floor beam just inside of the lower chord and also to the horizontal connecting plate of the lateral system which is attached to the bottom of the post. The effect of this construction is to cause a negative bending moment in the floor beam which neutralizes a part of the positive bending moments due to the dead and live loads. (See Fig. 78.)

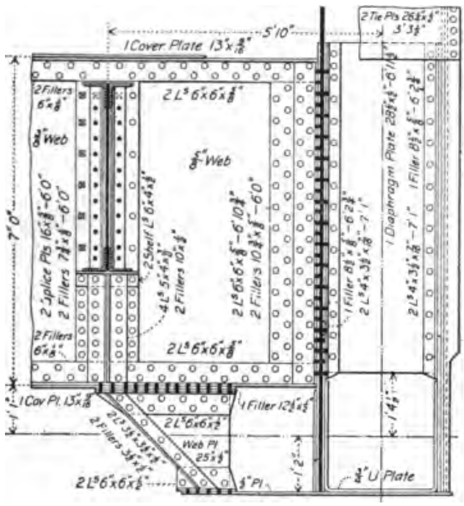


Fig. 78.

The floor beam of a deck bridge is shown on Plate V. The upper corner of the web plate is cut away to clear the diagonal eye-bars of the truss. The stiffeners below the stringers are required to distribute the concentrated loads to the web of the floor beam, fillers being put under the angles. An example in which the top of the floor beam is level with the top of the upper chord is given in Engineering Record, vol. 41, page 126, Feb. 10, 1900.

In double-track bridges the floor-beam flanges may be increased by means of side plates, as in plate girders. In the Bellefontaine, the Alton, and the Delaware river bridges this arrangement is adopted for the upper flanges only, while in the Rankin bridge it is adopted for both flanges. See Engineering Record, vol. 44, page 467, Nov. 16, 1901.

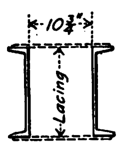
When a floor beam is not riveted to a post, but to some plates or to a short member which resembles a post in construction, but connects with a tension member, like the sub-vertical in a Baltimore or in a Pettit truss, or the suspender of a Pratt truss, as in Fig. 111, Art. 82, the floor beam is effectually stayed against rotation by rods extending to the adjacent panel points. The connection of a floor beam with the extension of a post below the lower chord is illustrated in Railroad Gazette, vol. 25, page 651, Sept. 1, 1893.

In all of these examples a diaphragm is required in order to carry its share of the load from the floor beam to the outer half of the post. It consists of a web plate united by a pair of angles to the two sides of the post.

Not many years ago end floor beams were employed in only a few cases, and those in trusses of large span. Now they are frequently used in short spans as well, and a number of railroads have adopted them as the standard construction.

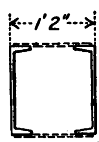
ART. 74. INTERMEDIATE POSTS.

The simplest form of post consists of two channels, whose flanges are united by short plates at or near the ends, called tie plates, and by lattice bars between. When the flanges are



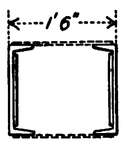
2–15″ [s 33 lbs.

Fig. 79.



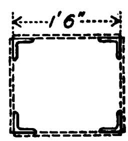
2–15″ [s 33 lbs.

Fig. 80.



2–15″ [s 48.5 lbs.
2 Pls. 12″×½″

Fig. 81.

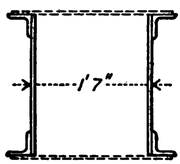


4 Ls 3½″×3½″×½″

Fig. 82.

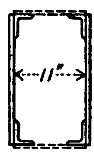
turned out (Fig. 79) as in the older practice, it is necessary to cut the channel flanges near the joints, as indicated in Fig. 111, Art. 82. When the flanges are turned in, as in Fig. 80, this cutting may be avoided and a stronger column secured for the

same out-to-out measurements. When the largest channels do not furnish sufficient area, the section is sometimes increased by adding two plates, preferably on the inside, as in Fig. 81. When still larger sections are required, the post is built up with plates and angles, as shown in Fig. 83. This form is sometimes said to consist of built channels. In Fig. 84 the angles are turned in, the advantage of so doing being the same as for rolled channels. The increasing area required for the posts toward the end of the span is obtained by increasing the thickness of the parts, or in case the thickness becomes excessive, by



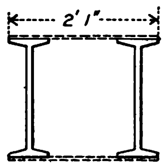
4 L^s 4"x4"x$\frac{11}{16}$"
2 Pls. 24"x$\frac{5}{8}$"

Fig. 83.



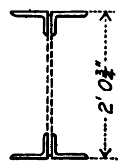
4L^s 4"x3"x$\frac{5}{8}$"
2 Pls. 22"x$\frac{13}{16}$"

Fig. 84.



2-20" I^s 90 lbs.

Fig. 85.



4 L^s 6"x4"x$\frac{3}{8}$"

Fig. 86.

adding an additional plate on each side, either of the full width of the side plates or to fill only the clear width between the angles.

The posts of the Victoria Jubilee bridge at Montreal have an unusual composition. Two I-beams are laced together (see Fig. 85) for each of the posts, 20-inch and 18-inch I-beams being employed in the posts near the ends and middle of a span respectively.

Fig. 84 shows how relatively narrow a post is sometimes made so as to be packed with the connecting diagonals in the upper chord. See Engineering Record, vol. 41, page 126, Feb. 10, 1900. On the other hand a post like Fig. 83, whose plates are only 22 inches wide, has the backs of the angles spaced 31¼ inches, in order to enter the outer spaces of the upper chord with its four webs. See Engineering Record, vol. 41,

page 516, June 2, 1900. Figs. 82 and 86 show additional post sections, which are mainly used for the sub-verticals of Baltimore and Pettit trusses, which support the upper chord midway between the long posts. The former section has also been used for collision struts.

Elevations of intermediate posts showing the tie plates and lattice bars which connect the two halves of the posts, as well as their diaphragms opposite the floor-beam connections, may be seen on Plates III, IV, and V, and in Fig. 111.

ART. 75. MAIN AND COUNTER DIAGONALS.

The simplest form used for a main tie consists of one or more pairs of eye-bars (Plate III). Tables of the standard sizes of eye-bars may be found in all of the handbooks. Sometimes, in order to secure stiffness in the panels of short spans requiring no counter bracing, the eye-bars are connected by riveting an angle to each bar and uniting the angles with lattice bars. In the panels which require counterbracing the same result is secured by using two pairs of angles laced together to form an I-section. (See Fig. 86 and Plate III.) In members with larger sectional areas a solid web plate is substituted for the lacing.

When the main ties are eye-bars the counters in the same panel consist either of an adjustable eye-bar, or of a square bar with loop eyes, when the required section is small. When laced angles are used for the main ties, the counters have the same composition.

Another method of securing greater stiffness has been adopted to some extent in which the counter ties are omitted and the main diagonals designed to take both tension and compression. The member is then made up either of two rolled channels laced together or of built-up channels, each one being composed of a web plate and two angles. The bridge over the Missouri river

at Bellefontaine, Mo., may be mentioned as a prominent example in which counterbraced diagonals are used, whose composition is the one mentioned last.

The larger vibration due to adjustable counters and the great difficulty in keeping them in proper adjustment has led to the design of the other forms, and so far as they have been compared under traffic, there is little or no difference between the action of Pratt trusses having counterbraced diagonals which take both tension and compression and those in which both main and counter ties are riveted members.

ART. 76. SUSPENDERS.

In the through Pratt truss the suspender or hip-vertical is the vertical tie which connects the upper end of the inclined end post and the second panel point of the lower chord. In the Baltimore and Pettit trusses there are not only the long suspenders, but a number of short ones whose duties are similar. These members have all the forms of section which were mentioned for the diagonals, whether counterbraced or not. If eye-bars are used, they are frequently connected by bent bars instead of by angles and the ordinary forms of lattice bars. (See Fig. 111, Art. 82.) When channels are employed, the flanges may either be turned in or out, and the same is true when the channel section is built up. The sectional area of built-up channels is increased sometimes by using double webs. When the I-section is used in a large truss, two flange plates are added to the two pairs of angles. For examples of the forms mentioned see Plates III and V, and Engineering Record, vol. 41, page 516, June 2, 1900, and vol. 37, page 384, April 2, 1898.

As the suspender in a through Pratt truss receives its stress only from loads in the first two panels, its stress changes more rapidly than that of any other member, and it also receives its

impact more directly. In order to reduce the excessive vibration thus produced some railroads require the suspender to be made of a riveted post section in all cases. This arrangement also prevents rising driftwood from buckling the floor and pulling the bridge off the pier.

In a deck Pratt truss with inclined end posts the only duty of the suspender is to support the lower chord members, and hence in this case it is made of a square bar with either upset or loop-welded eyes, or of two angles laced together so as to form a member about as wide transversely as the intermediate posts. The stiff member is preferable.

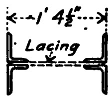
ART. 77. LOWER CHORD MEMBERS.

In simple pin-connected steel trusses the lower chord members are very seldom made of anything else than eye-bars, except in the two panels at each end. The depth of eye-bars used in trusses of ordinary span generally does not exceed 8 inches. On the other hand, the smaller depths are not now used to such a great extent as formerly, since it is considered desirable to use few comparatively heavy bars rather than a larger number of light ones. (See Plate IV.)

The largest eye-bars that have been used in any simple truss bridge in this country are those of the Delaware river bridge on the Pennsylvania Railroad, their depth being 12 inches, the greatest thickness $2\frac{1}{2}$ inches, and maximum finished weight of one bar, 56 feet long, 5500 pounds. Eye-bars 10 inches deep are used in the Louisville, Bellefontaine, Alton, and Rankin bridges, the greatest thickness being respectively $2\frac{1}{2}$, $2\frac{5}{8}$, $2\frac{11}{16}$, and $2\frac{1}{8}$ inches. In the Bellefontaine bridge the bars extend over two panels of the Baltimore trusses, being 55 feet long between centers of pins. In Fig. 111 are shown two pairs of eye-bars 51′ $3\frac{3}{4}''$ long, the inner ones being riveted to the

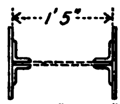
suspender and the outer ones resting on the horizontal legs of a pair of connecting angles.

In the best practice the lower chord members in the first two panels at each end of the span are designed to resist both ten-



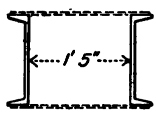
4 Ls 5"x3½"x⅜"

Fig. 87.



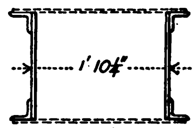
4 Ls 3½"x3½"x⅜"
2 Pls. 12"x½"

Fig. 88.



2–15" [s 45 lbs.

Fig. 89.

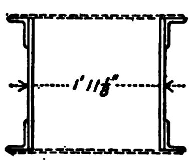


4 Ls 3½"x3½"x9/16"; 2 Pls. 18"x9/16"

Fig. 90.

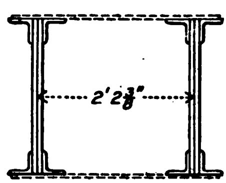
sion and compression. This construction enables the lower chord to resist the compression caused by the traction load when the brakes are applied to the train, or the thrust of a derailed car on the bridge, or that caused by a derailed car striking the end of the truss. It also reduces vibration, and increases the stiffness of the truss, especially in short spans. The principal forms of section are shown in Figs. 87 to 93 inclusive. Fig. 91 gives the section used in the end panels of the Alton bridge, and Fig. 92 those in the Bellefontaine bridge.

In a few cases the lower chord of pin-connected trusses is constructed with plates and angles from end to end. Fig. 93



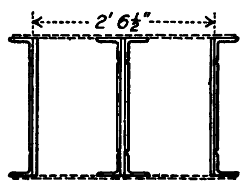
4 Ls 5"x3½"x½"
2 Pls. 22"x11/16":

Fig. 91.



8 Ls 4"x4"x⅝"; 4 Pls. 26"x¾"

Fig. 92.

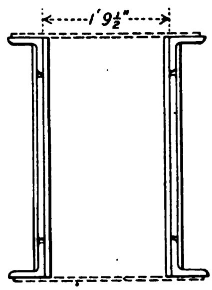


4 Ls 4"x4"x⅝"; 4 Ls 4"x4"x½"
3 Pls 23"x11/16"; 4 Pls. 14¾"x⅝"

Fig. 93.

gives the section in a panel toward the middle of one of the fixed spans of the United States bridge at Rock Island. In the end panels of the bridge only two webs are employed. Fig. 94

gives the section of the lower chord of the International bridge at Buffalo. The chord is made very deep in order to resist the flexure caused by the floor beams, which are spaced only half the distance between the panel points of the trusses. This construction was used to secure a shallow floor. The floor beams consist of 24-inch I-beams, and the stringers of 4 lines of 15-inch I-beams. See Engineering Record, vol. 43, page 567, June 15, 1901.



4 Ls 6" x 6" x 7/8"; 2 Pls. 40" x 7/8"
2 Pls. 27½" x 7/8"; 2 Pls. 38" x 7/16"

Fig. 94.

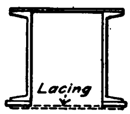
The bridge department of the Baltimore and Ohio Railroad has designed some spans in the vicinity of 150 feet in which the use of eye-bars is restricted to the end ties and the entire bottom chord, all the bars being laced together in order to eliminate as far as possible the vibration of these members. Sometimes the eye-bars in the end panels only are laced instead of using members composed of plates and shapes, as shown in Fig. 111, Art. 82. The use of bottom chords which are stiff throughout is also referred to in Chapter XI.

ART. 78. UPPER CHORD AND END POSTS.

One of the simplest sections of an upper chord member is shown in Fig. 95. The flats below the channels are used to balance the section about a horizontal axis passing through the centers of the channel webs. These are often omitted, but unbalanced sections are not regarded favorably by the best designers. When the section is so small that the required thickness of the metal is less than the minimum allowed, the cover plate and flats are omitted and then the top of the member is laced as well as the bottom.

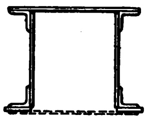
The compositions indicated in the two examples given in Figs 96 and 97 are much more frequently employed for ordi-

nary spans. In the one case the section is balanced by means of flats, while in the other the lower angles are increased in size for the same purpose. The former method is preferred, as it simplifies the construction at the joints where pin plates



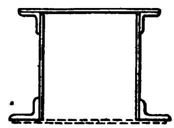
1 Cover Plate 19"×$\frac{3}{8}$"
2–15" [s 33 lbs.
2 Flats 4"×$\frac{3}{4}$"

Fig. 95.



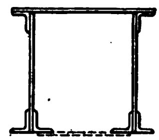
1 Cover Pl. 22"×$\frac{3}{8}$"
2 Web Pls. 16"×$\frac{3}{8}$"
4 Ls 3$\frac{1}{2}$"×3$\frac{1}{2}$"×$\frac{3}{8}$"
2 Flats 4$\frac{1}{2}$"×$\frac{3}{4}$"

Fig. 96.



1 Cover Plate 24"×$\frac{7}{16}$"
2 Web Plates 18"×$\frac{1}{2}$"
2 Angles 3$\frac{1}{2}$"×3$\frac{1}{2}$"×$\frac{3}{8}$"
2 Angles 5"×3$\frac{1}{2}$"×$\frac{7}{8}$"

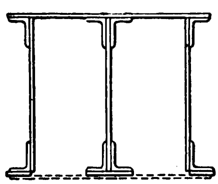
Fig. 97.



1 Cover Plate 24"×$\frac{7}{16}$"
2 Web Plates 20"×$\frac{5}{8}$"
4 Angles 3$\frac{1}{2}$"×3"×$\frac{7}{16}$"
2 Ls (inside) 3$\frac{1}{2}$"×3$\frac{1}{2}$"×$\frac{1}{2}$"

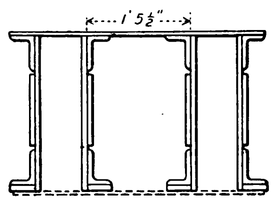
Fig. 98.

must be attached to the sides in order to secure sufficient bearing on the pins. In Fig. 98 the section is balanced by using two angles instead of one at the bottom of each web plate. At the panel points the horizontal legs of the inner angles are cut to afford the necessary clearance for the posts and diagonals. The latticing is connected to the inner angles only. This section is taken from the Northern Pacific Railway's standard plan for a 200-foot through pin bridge. (See Plate IV.)



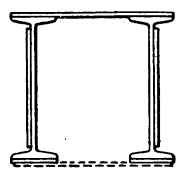
1 Cover Plate 34"×$\frac{3}{8}$"
2 Web Plates 26"×$\frac{9}{16}$"
1 Web Plate 26"×$\frac{3}{4}$"
8 Angles 5"×3$\frac{1}{2}$"×$\frac{7}{16}$"
2 Flats 4"×$\frac{5}{8}$"
1 Flat 8"×$\frac{5}{8}$"

Fig. 99.



1 Cover Plate 43"×$\frac{1}{2}$"
2 Web Plates (inside) 25$\frac{1}{4}$"×$\frac{7}{8}$"
2 Web Plates (outside) 25$\frac{1}{4}$"×$\frac{3}{4}$"
4 Plates 12"×$\frac{3}{4}$"
8 Angles 6"×4"×$\frac{3}{4}$"
4 Flats 4"×1"

Fig. 100.



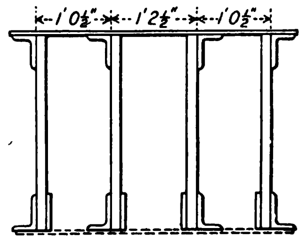
1 Cover Plate 27$\frac{1}{2}$"×$\frac{5}{8}$"
2–24" I-Beams
100 lbs
2 Web Plates 20×$\frac{5}{8}$"
2 Flats 7$\frac{1}{2}$"×$\frac{5}{8}$"

Fig. 101.

Additional area is obtained not only by increasing the thickness of the plates and shapes, but also by putting additional

web plates in the clear space between the angles or by placing a web plate of the full depth inside of each of the others. Fig. 99 shows a section containing three webs, and in this case also the outer webs are strengthened in the manner just described. The maximum upper chord section of the Bellefontaine bridge is given in Fig. 100. That of the Delaware river bridge is similar to this except that the inner upper angles are placed on the outside of the inner webs as indicated on Plate V, which shows some details of another bridge on the same division of the Pennsylvania Railroad.

Fig. 102 gives the composition of the largest section of the upper chord of the Monongahela river bridge at Rankin, Pa., its sectional area being 334.52 square inches. It is the largest chord section of any simple truss in existence. It will be noticed that the flats are placed opposite the vertical legs of the angles instead of being riveted to their horizontal legs. The chords of the heavy truss in the Monongahela river bridge at Port Perry, Pa., are a little wider, but the depth and area are less. The composition is as follows: 1 cover plate, $50'' \times \frac{5}{8}''$; 2 pairs of outer web plates, $30'' \times \frac{13}{16}''$; 2 pairs of inner web plates, $30'' \times \frac{5}{8}''$; 4 upper angles, $4'' \times 4'' \times \frac{5}{8}''$; 2 outer lower angles, $6'' \times 4'' \times \frac{3}{4}''$; 2 inner lower angles, $6'' \times 6'' \times \frac{7}{8}''$; 2 outer flats, $6'' \times \frac{1}{2}''$; and 2 pairs of inner flats, $6'' \times \frac{5}{8}''$. The arrangement of the shapes is similar to that in Fig. 102, except that the outer flats are placed between the outer angles and the web plates. Five intermediate lines of rivets, with a large pitch, are used to connect the several pairs of web plates. The light truss in the same bridge has only three webs. In both bridges the



1 Cover Plate $48'' \times \frac{5}{8}''$
4 Web Plates $33'' \times 1\frac{3}{4}''$
8 Angles $6'' \times 4'' \times \frac{3}{4}''$
4 Flats $6'' \times \frac{3}{4}''$

Fig. 102.

ends of the chords and end posts where pin bearing is required have short angles placed opposite the upper angles and extended the full length of the pin plates.

The new trusses of the International bridge at Buffalo, erected in 1901, have upper chords of a very unusual section, shown in Fig. 101. Toward the ends of the span the side plates are reduced, and finally omitted. The lacing at the bottom consists of $3\frac{1}{2}'' \times 2\frac{1}{2}'' \times \frac{3}{8}''$ angles. At the panel points portions of the inner flanges of the I-beams are cut away to provide the clearance needed to pack the web members.

When the chords and end posts have either three or four webs, it is important that their ends be prevented from shifting their relative position after the pin holes are bored, or else trouble is caused in erection. The same conditions apply to the sections where the chords are spliced. This is accomplished by means of transverse diaphragms, as indicated on Plate V. It will be noticed that between the inner and outer webs the diaphragms consist simply of two angles, while between the inner webs plates are also used.

The construction of end posts is usually the same as that of the chords in the same span, the variations rarely being more than those between the upper chord members in different panels. Occasionally the width of the end post may be different from that of the upper chord, but this is rather exceptional.

ART. 79. LATERAL BRACING.

Formerly the upper lateral ties of through bridges consisted of adjustable square bars or round rods connected either to the top of the upper chord or to the middle of its inner web by means of connecting plates and pins. In long spans two sets of ties were often used connected to the top and bottom of the chord respectively. This construction is now seldom employed,

nearly all the standard specifications for railroad bridges stating that stiff members are preferred for the lateral bracing. Those who still use the adjustable members claim that they are not only much lighter, but that the upper chord can be more thoroughly lined up by this means. The object of the stiff laterals is to secure greater lateral stiffness in the bridge, as well as to avoid the difficulty of maintaining the rods in proper adjustment. Many specifications state that it is preferable to avoid altogether the use of adjustable members in trusses, lateral and sway bracing.

Stiff lateral diagonals are most frequently composed of single angles as illustrated in Plate III, and Fig. 111. Sometimes two angles placed, back to back are employed. In order to give greater vertical stiffness to these members a section like Fig. 86 is used, consisting of two pairs of angles laced together, the depth of the section being equal to that of the upper chord so that the connection with it may be made on both top and bottom. (See Plate VII , Chap. XI.) Laterals of this type are used in the Delaware river bridge. Occasionally in short spans the composition is modified by latticing two single angles instead of two pairs of angles. This form is used in the Monongahela river bridge at Rankin, Pa., the size of both angles being $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$. The long span over the same river at Port Perry has adjustable rods.

The various sections described are used also for lower laterals of through bridges. In many cases where adjustable laterals are still used in the upper system, stiff members are employed in the lower system. (See Plate IV.) This statement also applies to the bridge at Port Perry, whose lower laterals consist of two pairs of angles latticed together. As these laterals are not connected to the stringers, they are stiffened in a horizontal direction by means of four horizontal members of similar composition which are connected at their extremities to the laterals

at their quarter points, thus forming a rectangle in plan The laterals stiffen each other also by the connection at their centers. Attention is called to the forms of splices used for both upper and lower laterals on Plates III and VII.

The connections of the lower laterals to through trusses is often very eccentric, causing large horizontal bending moments in the ends of the floor beams. This is avoided, in the best designs, by using larger connecting plates, and by incurring the cost of somewhat greater inconvenience in field riveting. In the upper lateral system the effect of eccentricity is not so serious, since the stresses are smaller and the connection is made to the stiff upper chord. Let the student observe the character of the lateral connections in this respect on Plates III, IV, VI , and VII .

The construction of the upper lateral system in deck bridges is practically the same as that of the lower system in through bridges, and that of the lower system of deck bridges the same as that of the upper system of through bridges. Sometimes the lower laterals are omitted in alternate panels, while in other cases they are omitted entirely. The latter arrangement is adopted in the standard plans for pin-connected deck bridges on the Northern Pacific Railway.

The lateral struts which are perpendicular to the upper chords of through trusses form also a part of the transverse or sway bracing. Sometimes the rest of the sway bracing consists merely of brackets connecting the lateral struts to the posts of the trusses, while at other times this is connected to a lower or intermediate strut by means of two or more web members as shown in the next article. In short spans the lateral strut is composed of two pairs of angles placed back to back and laced together as in Fig. 86, its depth being equal to that of the upper chord to whose upper and lower flanges it

is riveted by connecting plates. (See Plates III and VII.) Occasionally the upper angles are placed with their horizontal flanges on the lower side, extended across the top of the chord and riveted directly to it. Where the upper chord is rather deep and the trusses are separated by double tracks, the angles are often placed in the corners of a rectangle as in Fig. 82, Art. 74, and laced on the four sides. Two channels laced together are occasionally used. Another form of section is that in which the lacing of the first form mentioned is replaced by a solid web, forming practically a small plate girder.

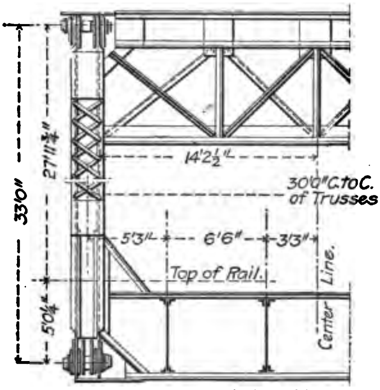
When the web connections of the sway bracing are rather close together, the lateral strut is sometimes reduced to a single pair of angles (Plate IV) or to one pair of angles with a web plate between, the latter form being shown on Plate VII . In double-track bridges this section is increased in stiffness horizontally by using bulb angles instead of the ordinary angles.

The composition of lower lateral struts in deck bridges comprises all the forms mentioned above except those containing a solid web plate with either one or two pairs of flange angles.

ART. 80. PORTAL AND SWAY BRACING.

When the required clearance extends to within two or three feet of the top of the lateral strut, the intermediate sway bracing of a through bridge consists merely in connecting the strut to the post at each end by means of a bracket or knee brace. (See Plates III and VI .) When there is more head room one of the simplest styles of bracing consists of a lattice girder, with a double system of webbing, as shown on Plate IV. The lower flange is placed as low as the head room will allow. With increasing depth four systems of webbing may be used, an example of which is given on Plate VII . For other examples, see the inset of the Engineering News, Jan. 11, 1900. Where

the depth is large, the lower strut is sometimes made like the upper or lateral strut. It will be noticed that the bracing on Plate VII also contains a small bracket. The use of brackets is generally confined to cases where the depth is small.



Section through L3. U2.

THE ENGINEERING RECORD.

Fig. 103.

Another type is shown in outline in Fig. 103, and its details are given in Fig. 104. Sometimes the verticals are omitted in the webbing, thus reducing it to the Warren type of truss. The number of panels depends on the depth of the bracing and on the width of the bridge. An example of this form may be seen in Engineering Record, vol. 37, page 386, April 2, 1898.

The small connecting plates shown in the top view and section are intended to connect with a longitudinal strut which helps to stiffen the lateral struts in a horizontal direction, since it is also attached to the lateral diagonals at their intersection.

Fig. 105 shows two forms of intermediate sway bracing, one between the long posts of the trusses in which a quadruple system of diagonals is used, and the other between the sub-vertical struts with only two diagonals. In both cases the upper and lower struts are composed of a plate and a pair of bulb angles. In some cases the single pair of diagonals is used throughout the span, and occasionally a sub-vertical is suspended from the intersection of the diagonals to support the center of the lower strut. With further increase in depth the sway bracing is sometimes divided into two panels, one above the other, by means of an intermediate horizontal strut. In the Engineering

Record, vol. 44, page 467, Nov. 16, 1901, may be found an illustration of the sway bracing at the middle of the span of the Rankin bridge. The lateral strut consists of two pairs of angles $6'' \times 4'' \times \frac{3}{8}''$, connected by a system of double intersection lacing with $3'' \times 2'' \times \frac{5}{16}''$ angles; the other two struts consist

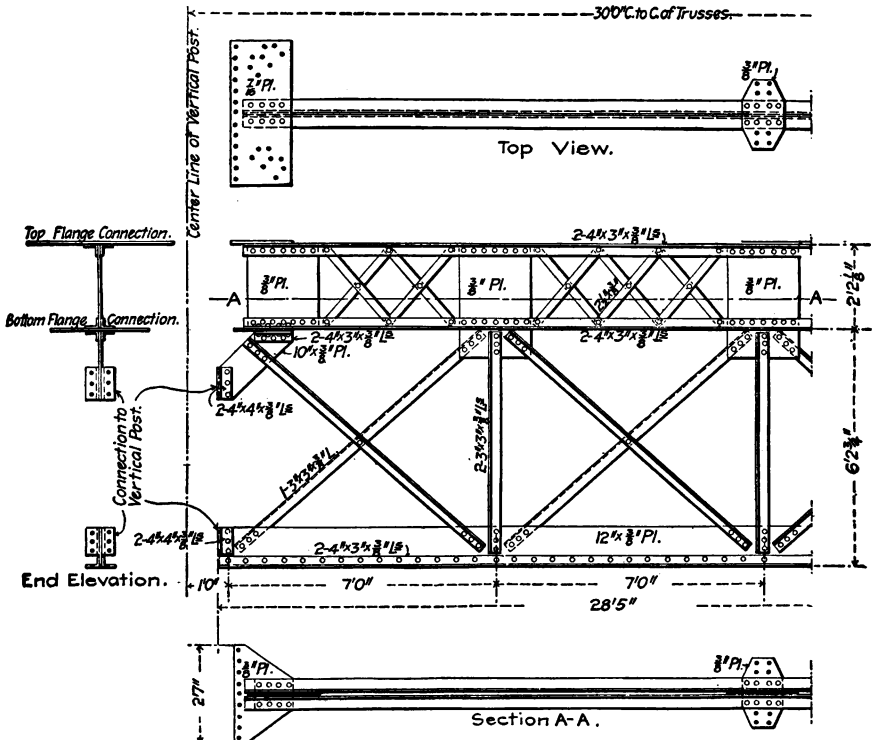


Fig. 104.

of two pairs of $5'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ angles laced with bars so as to be 18 inches deep, and the two diagonals in each panel are composed of two $3'' \times 3'' \times \frac{3}{8}''$ angles placed back to back. Toward the end of the span the bracing has only one panel, and at some intermediate points a single pair of diagonals crosses the two panels intersecting the middle strut at its center.

In the shorter spans of the Victoria Jubilee bridge at Montreal the upper strut is made up of two pairs of $6'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ angles laced, the lower strut of one web plate, $10'' \times \frac{3}{8}''$, and two flange angles, $6'' \times 3\frac{1}{2}''$, each of the two intersecting diagonals of one angle, $4'' \times 3'' \times \frac{3}{8}''$, and the sub-vertical of one angle, $3'' \times 3'' \times \frac{3}{8}''$. In the long span the composition is the same except that the angles and plate are increased in size.

The general character of the sway bracing of deck bridges is about the same as for through bridges. One example of both the intermediate and the end sway bracing is shown on Plate V. Another example may be found on the inset of the Engineering News, Nov. 29, 1900, and a third one in the Engineering Record, vol. 41, pages 125 and 126, Feb. 10, 1900.

Adjustable rods are still used to some extent in the sway bracing of both through and deck bridges, but the practice is not generally regarded with favor.

A number of the forms employed for intermediate sway bracing are also used in portal bracing, the details being made stronger, however, on account of the greater duty of the latter. Plate IV shows a portal having flanges with unequal-legged angles of ample size, and with deep plates to receive the connections of the web members, which consist of two systems of diagonals. The wide plates are continued around the ends of the portal, and extended into the bracket, so as to make a very rigid connection with the inner sides of the end posts. As indicated on the plate, this is a standard design of the Northern Pacific Railway. In the reference to Engineering Record mentioned in the preceding paragraph may be seen the view and details of a portal only about $4\frac{1}{2}$ feet deep at the middle, with double in tersection webbing. The lower flange is curved down at the ends to form the flanges of the brackets, and solid web plates form the bracing in the end panels. The standard portal of the

Fig. 105. Double-track Through Bridge over the Missouri River at Bellefontaine, Mo.

New York Central and Hudson River Railroad is given on Plate VII , Chap. XI.

In some cases where the head room is limited the portal bracing consists practically of two complete double intersection lattice girders with their connecting end brackets, one riveted to the top and the other to the bottom flanges of the end posts, the corresponding flanges of both girders being united by lacing. (See Fig. 6, Art. 3.) A plate is sometimes substituted for the upper lacing. An example of a double portal bracing, but of somewhat different design, is shown on Plate VI . Under similar conditions of limited head room the portal bracing is occasionally composed simply of a plate girder and of brackets with solid webs.

Perhaps the best illustration of the application of a lattice portal bracing to a bridge of long span is that of the Bellefontaine bridge shown in Fig. 105. The top strut consists of a web plate $30'' \times \frac{1}{2}''$ and two bulb angles $9'' \times 3\frac{1}{2}'' \times \frac{7}{16}''$, the lower strut of one plate $27'' \times \frac{1}{2}''$, one angle $4'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$, and one bulb angle $9'' \times 3\frac{1}{2}'' \times \frac{7}{16}''$, and each of the twenty diagonals of one angle $5'' \times 5'' \times \frac{1}{2}''$ angle. The plates extend around both sides of the bracing similar to that on Plate VII , and with neatly rounded corners.

The present practice in the design of portals for bridges whose depth affords adequate room consists in using relatively few members with sufficient strength to secure that degree of lateral rigidity which is now regarded as so essential. The members are all made of the same depth as the end posts, so as to permit them to be riveted to both the top and bottom flanges of the end posts. An excellent example of such a design is the portal of the United States bridge at Rock Island, Ill. The view given in Fig. 106 is that of the portal of the draw span, but it has the same construction as those used on the fixed spans. In the $216\frac{1}{2}$-foot fixed spans the lower strut has one

Fig. 106. Portal of United States Bridge at Rock Island, Ill.

Fig. 107. Portal of Delaware River Bridge, Philadelphia.

cover plate $16\frac{1}{2}'' \times \frac{3}{8}''$, four angles $3'' \times 3'' \times \frac{3}{8}''$, and is laced on three sides. The diagonals have two pairs of angles $5'' \times 3'' \times \frac{3}{8}''$, with one line of lacing. The upper strut has an upper cover plate $17\frac{1}{4}'' \times \frac{3}{8}''$, three angles $3'' \times 3'' \times \frac{3}{8}''$, one angle $4'' \times 4'' \times \frac{3}{8}''$, and a lower cover plate $7'' \times \frac{3}{8}''$. It is laced on two sides, one side being perpendicular to the flanges of the end post, and the other in the plane of the beveled end of the end post. The provision of connecting plates with curved edges indicate that some attention was paid to æsthetic considerations in this design.

The portal of the Delaware river bridge near Philadelphia is divided into two panels, one above the other. Fig. 107 indicates that the lower strut is practically a plate girder whose depth equals that of the end posts, while the middle strut and the diagonals consist of two pairs of angles laced together. The top strut is of novel design. In composition it resembles that of an upper chord member, but the two web plates are respectively perpendicular to the flanges of the end post and of the upper chord, and both the cover plate and the lower lacing are bent to the angle made by the end post with the adjoining upper chord member. Square connections could thus be made on one side with the portal diagonals and on the other side with the top laterals, which also consist of two pairs of angles laced as deep as the chords.

Another portal containing some new details is that of the Union Railroad Bridge at Rankin, Pa., shown in Fig. 108. Both the upper and the lower struts consist practically of two plate girders whose flanges, each having only one angle, are extended across the end posts, and riveted to them on the upper and lower sides respectively. The girders have their corresponding flanges laced together with a single system of diagonals composed of single angles. Double triangular brackets with solid webs and connecting plates are also used. In addition to the diagonals of the portal, a strut of the same composition as

the diagonals connects the middle of each horizontal strut with the intersection of the diagonals.

The portal bracing of the bridge erected by the same railroad at Port Perry differs from this one by substituting for the strut

Fig. 108. Portal of Rankin Bridge.

just mentioned one of only two angles laced in a similar way, but extending horizontally across the intersection of the diagonals, and riveted at each end to the top and bottom of the end post. The upper strut is also different in containing only four angles laced on the four sides.

This bridge contains the unusual feature of a double plate-girder portal bracing, connecting the feet of the end posts on their top and bottom flanges. The web plates of each of these girders are not continuous, but are connected by angles to the webs of the stringers, and thus to each other. The flanges, however, are continuous and are field-riveted to the webs. They consist of single angles. The function of this bracing is performed in many other bridges by an end floor beam riveted to the end posts.

The composition of the sway and portal bracing of the Victoria Jubilee bridge is given in Engineering Record, vol. 38, page 488, Nov. 5, 1898. Each of these contains only two struts and two intersecting diagonals.

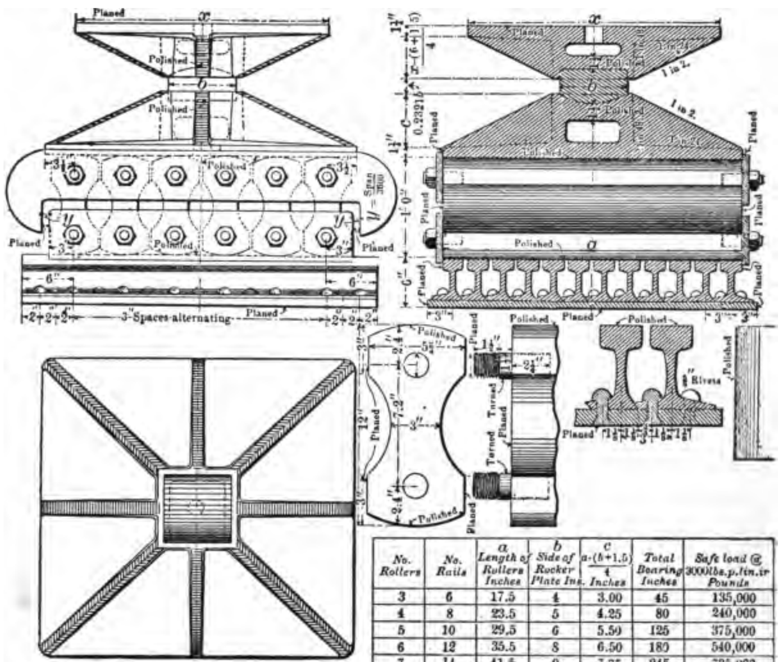
ART. 81. EXPANSION BEARINGS.

Pedestals, friction rollers, and bed plates, similar to those described in Art. 44, are used also for truss bridges. Two examples of expansion bearings containing cylindrical rollers are given on Plate III and in Fig. 111.

Complete detail drawings of pedestals, nests of cylindrical rollers, and rail plates, together with the castings for the fixed bearings, may be found on the insets of the Engineering News for Jan. 5 and Feb. 2, 1899. The nests contain 7 rollers each, their diameters being $4\frac{1}{2}$ and $4\frac{7}{8}$ inches respectively.

Fig. 109 shows the details of a standard expansion bearing, designed by GEORGE S. MORISON, which contains some valuable improvements over those employed in Europe. The steel rollers are 12 inches in diameter and spaced 6 inches between centers. The sides are parallel near the top and bottom, and hollowed out along the middle to facilitate cleaning with a brush. Contact between the parallel sides of the rollers prevents them from tipping over, but an additional provision against it is afforded

by means of the side plates, which engage stud bolts screwed into the ends of the rollers. The clearance between the hook of the upper plate and the square ends of the lower plate allows a linear movement of $y = \text{span}/3000$ in both directions from the mean position. The rollers rest on a rail plate consisting of



No. Rollers	No. Rails	a Length of Rollers Inches	b Side of Rocker Plate Ins.	c $\frac{a-(b+1.5)}{4}$ Inches	Total Bearing Inches	Safe load @ 3000lbs. p. lin. in Pounds
3	6	17.5	4	3.00	45	135,000
4	8	23.5	5	4.25	80	240,000
5	10	29.5	6	5.50	125	375,000
6	12	35.5	8	6.50	180	540,000
7	14	41.5	9	7.75	245	735,000
8	16	47.5	10	9.00	320	960,000
9	18	53.5	11	10.25	405	1215,000
10	20	59.5	12	11.50	500	1500,000
11	22	65.5	13	12.75	605	1815,000
12	24	71.5	14	14.00	720	2160,000

Fig. 109.

T-rails riveted to a plate, with their tops afterward planed and polished. In large bridges the rail plate is bolted to the cast base, which is directly supported by the pier masonry. As the dust accumulates it is readily removed by passing a long-handled brush between the rails.

On top of the rollers rests a steel casting with its lower surface polished, and this in turn supports another casting by means of a block of polished steel called a rocker plate. The rocker plate fits into a socket in each casting, the surfaces of contact being segments of horizontal circular cylinders, whose axes are respectively parallel and perpendicular to the direction of the rollers. The radius of curvature in each case equals the length of one side of the square rocker plate. The upper casting sustains the pedestal, which in turn supports the end pin of the truss, and the connecting bolts pass also through the flanges of the stiff lower chord. The object of the rocker plate is to allow the bridge to adjust itself when erected, so that the bearing on the rollers, and hence also that on the bed plate, may be uniform. This eliminates the unequal distribution of load, which would otherwise be caused by imperfect workmanship in the construction of the truss and its supports. To secure the transverse stiffness of the lower ends of the end posts, they are preferably connected by an end floor beam which is riveted to them after the bridge is swung. The side plates project above and below the rollers respectively, thus acting as guides to prevent any lateral movement of rollers or casting.

In the vertical line of dimensions in Fig. 109 the next to the highest one should read $\frac{x-(b+1.5)}{4}$, while the value $0.2321\,b$ does not belong to c, but to the dimension directly above c. The safe load given in the table is that recommended by the designer, no addition being made to the live load for impact. The allowance for impact is included in the unit stress adopted. See Transactions of American Society of Mechanical Engineers, vol. 15, page 153, 1894, and vol. 16, page 724, 1895, for an account of the evolution of this bearing and some illustrations of its application. The description and detail drawings of a modified form of this bearing, in which a pin casting takes the place of

the usual bolster and of the top casting and rocker plate, thus materially reducing the height required, may also be found in Engineering Record, vol. 32, page 93, July 6, 1895.

In order to avoid the danger of the rollers getting out of place under the frequent jars to which the lighter bridges are subject, another improvement has been added by fitting steel plates into grooves cut in the ends of the middle roller, the plates projecting beyond the surface of the roller and forming teeth to engage spaces cut into the rail plate below and the bearing plate above. The details of this device may be seen on Plate II.

A side elevation of the fixed and expansion bearings of the Davenport, Rock Island and Northwestern Railroad bridge at Rock Island, Ill., may be seen in the inset of Engineering News, Jan. 11, 1900. This is a different type of bearing from the standard described above. The I-beams extend under both bearings over the full width of the pier.

The 6-inch segmental rollers of the International bridge at Buffalo are illustrated in Engineering Record, vol. 43, page 567, June 15, 1901. They are 3 inches wide, but have cylindrical spaces 6 inches long cored out of their sides and separated by $\frac{3}{4}$-inch vertical webs. The rollers are 3 feet 5 inches long.

Fig. 110 gives the details of the 12-inch rockers with parallel sides used in the truss shown on Plate V. The center roller has a spur or gear tooth at each end on both top and bottom, and these enter slots in the roller bed plate and the shoe respectively, thus retaining the rollers in their proper position. Grooves in the centers of the rollers engage longitudinal center strips to prevent the trusses from shifting sideways. The figure also indicates the construction of the pedestal and bed plate at the fixed end of the span.

In the Delaware river bridge seven segmental cast-steel rollers 18 inches in diameter are used to take care of a truss

reaction of 1200 tons. The rollers are of the same type as those mentioned in the preceding paragraph, and are $8\frac{1}{2}$ inches wide and 8 feet $2\frac{1}{2}$ inches long. The gear teeth on the middle roller are 7 inches long. A view of the fixed and expansion bearings on one pier is shown in Engineering Record, vol. 40, page 596, Nov. 25, 1899.

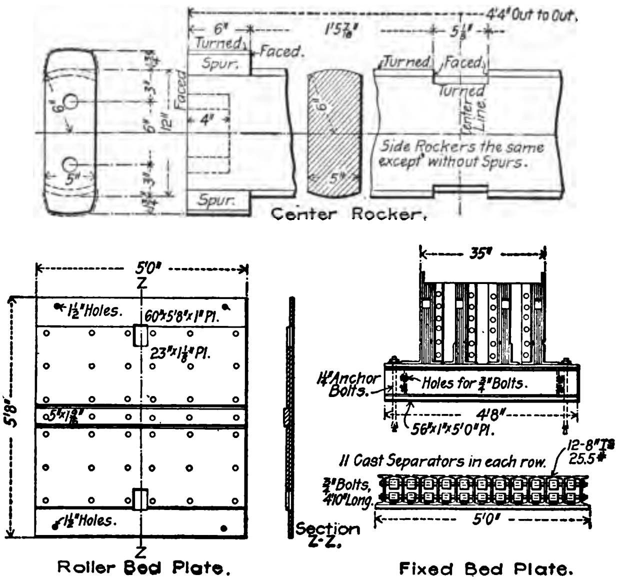
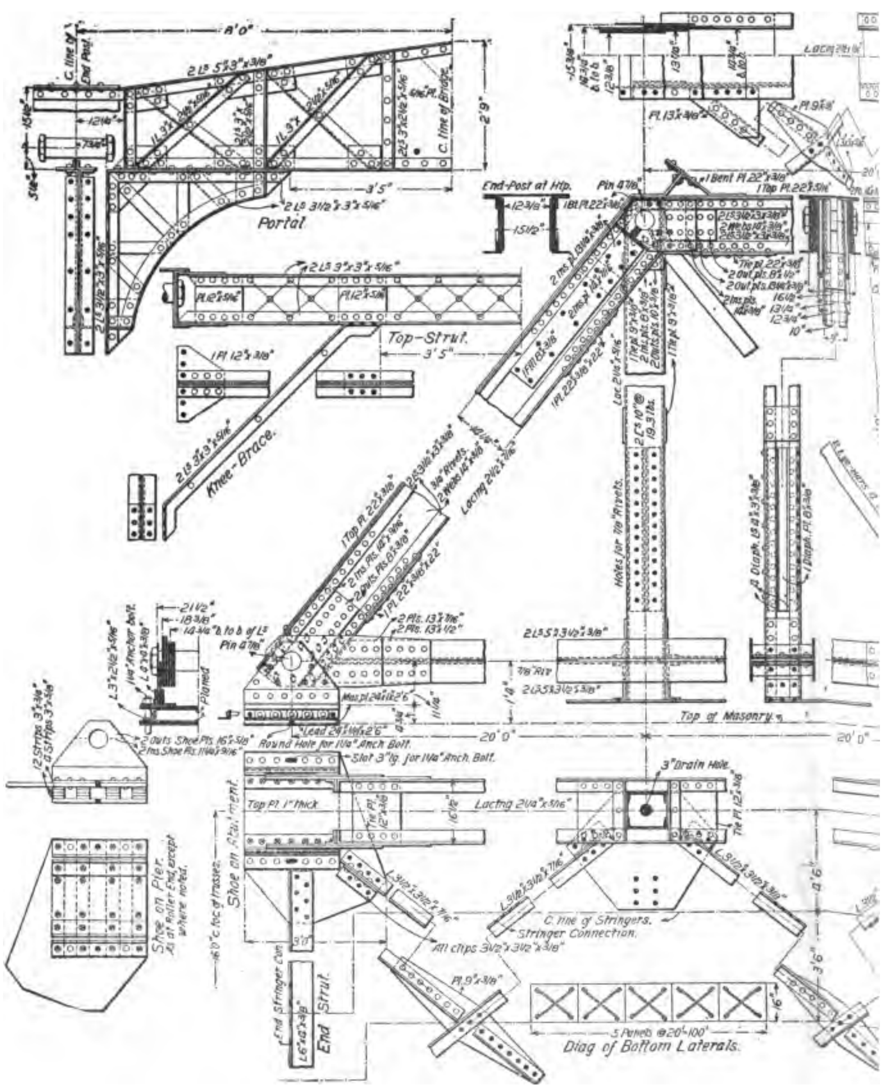


Fig. 110.

ART. 82. RAILROAD PIN BRIDGES — REFERENCES.

Variations in the composition of members throughout the span, the relations between the forms and dimensions of connecting members, and many of the smaller details related to the connections at the joints can best be studied by consulting the general drawings of different trusses. The following references are given to enable the student to become familiar with recent practice in these respects, no reference being included whose date precedes 1895, and only a few that are earlier than 1898.

Bridge No. 14, Midland Division, Bal

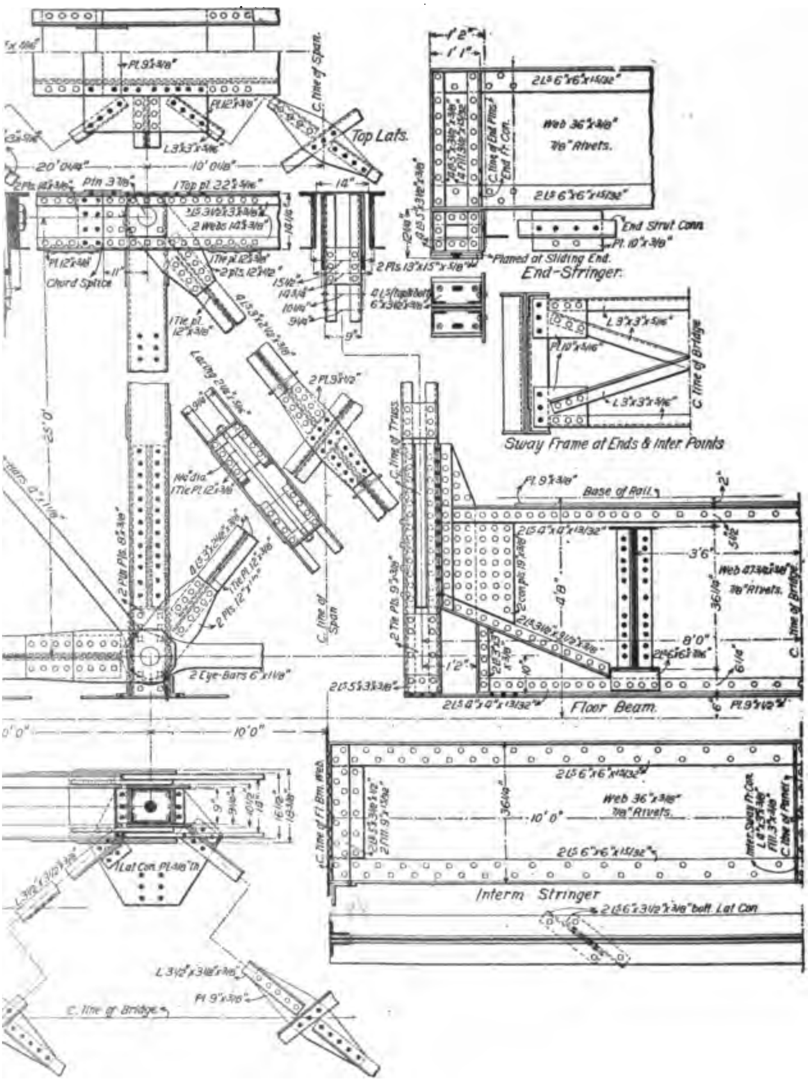


DETAILS OF A SINGLE-TRACK THROUGH BRIDGE OVE

SPAN, 100 FEET. ERECTED

See references to separate details in Arts.

Baltimore and Ohio Railroad.



·VER DEER CREEK, EAST OF MT. STERLING, OHIO.

:ED IN DECEMBER, 1894.

:ts. 71, 73, 74, 75, 76, 79, 80, and 81.

Recent small bridges on the Baltimore and Ohio Railroad. Railroad Gazette, vol. 27, page 34, Jan. 18, 1895.

General plan of Bridge No. 14, Midland Division. (See Plate III.) Single-track through Pratt-truss bridge, span 100 feet. The article contains extracts from the specifications which indicate the character of the structures.

Bridge Work on the Baltimore and Ohio Railroad. Engineering Record, vol. 41, page 271, March 24, 1900.

A description of the characteristic features of short-span bridges designed by the bridge department for extensive recent improvements where old bridges were replaced by heavier and stiffer structures. Several views, but no drawings.

Standard Plans for 120-foot Pony-truss Bridges, Northern Pacific Railway. Engineering News, vol. 41, page 14, Jan. 5, 1899. Standard Plans for 130-foot Through Truss Bridges, Northern Pacific Railway. Engineering News, vol. 41, page 69, Feb. 2, 1899.

Complete detail drawings. These standards have been superseded by those referred to below, but they will furnish the student a good opportunity for comparative study. The 120-foot truss is replaced in the new standards by a riveted truss.

A Trunk Line Deck Bridge. Engineering Record, vol. 33, page 58, Dec. 28, 1895.

Span, 127′ 11⅝″. Partial plans of a single-track deck bridge across Big Pipe Creek on the New York, Lake Erie and Western Railroad.

Short-span Railroad Bridges. Engineering Record, vol. 40, page 717, Dec. 30, 1899.

Extracts from specifications giving unit stresses; description of a few special details of spans of 125 and 150 feet of the Oregon Railroad and Navigation Co.; illustrations of floor-beam connection to the hanger at the first panel point, and of templets for reaming the connections of the floor system.

The Terminal Improvements of the Chesapeake and Ohio Railway, at Richmond, Va. Engineering News, vol. 44, page 379, Nov. 29, 1900.

Span, 133′ 7⅛″. The drawings show the details of one of the deck spans of the river viaduct.

Northern Pacific Standard Bridge Plans. By RALPH MODJESKI. Journal of Western Society of Engineers, vol. 6, page 51, Feb., 1901.

The plates accompanying this paper include the general plans of a 160-foot deck span and of a 200-foot through span, both being single-track bridges. (See Plate IV.)

Special Bridge and Viaduct Construction in Western Pennsylvania. Engineering Record, vol. 41, page 465, May 19, 1900.

Span, 177′ 3″. Description and illustration of some of the principal details of the Union Railroad bridge which crosses the Pittsburg, Virginia and Charleston Railroad at Port Perry, Pa. (See illustrations of details in Figs. 77, 103, 104, and 111.)

The Lehigh Valley Railroad Bridge at Easton, Pa. Engineering Record, vol. 41, page 124, Feb. 10, 1900.

Description and drawings showing the details of the outside trusses of two of the spans in which the track is on an eight-degree curve. One is a through span 187 feet long, and the other a deck span 215′ 1½″ long. There is also a general view of the bridge and its approach. The bridge is noted for the special features of construction required by its difficulties of alignment, grade, etc.

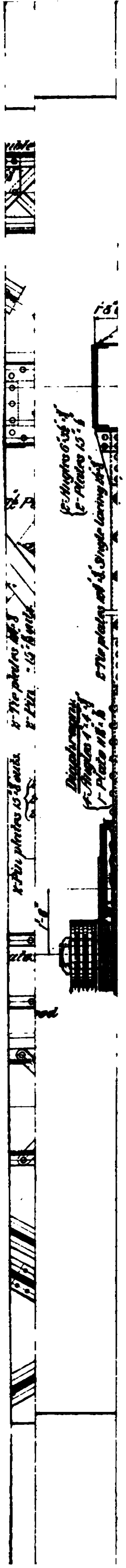
The Newport and Cincinnati Bridge. Engineering Record, vol. 37, page 448, April 23, 1898.

Span, 198′ 6″. Partial plans showing the details of one of the short Pratt truss through spans of this four-truss bridge. The upper chord is curved. The bridge accommodates a single-track railroad, two street railroad tracks, a roadway, and a sidewalk.

The New United States Rock Island Bridge. Engineering Record, vol. 37, page 384, April 2, 1898.

Span, 216′ 6¾″. Description and detail drawing of one of the ten-panel Baltimore trusses of a double-deck through bridge. The double-track railroad is on the upper deck. The portal and intermediate sway bracing of the 258-foot span are also shown on small-scale sections. (See also Engineering News, vol. 36, page 406, Dec. 17, 1896.)

Bridge 69, New York Division, Pennsylvania Railroad. Engineering Record, vol. 39, page 371, March 25, 1899.



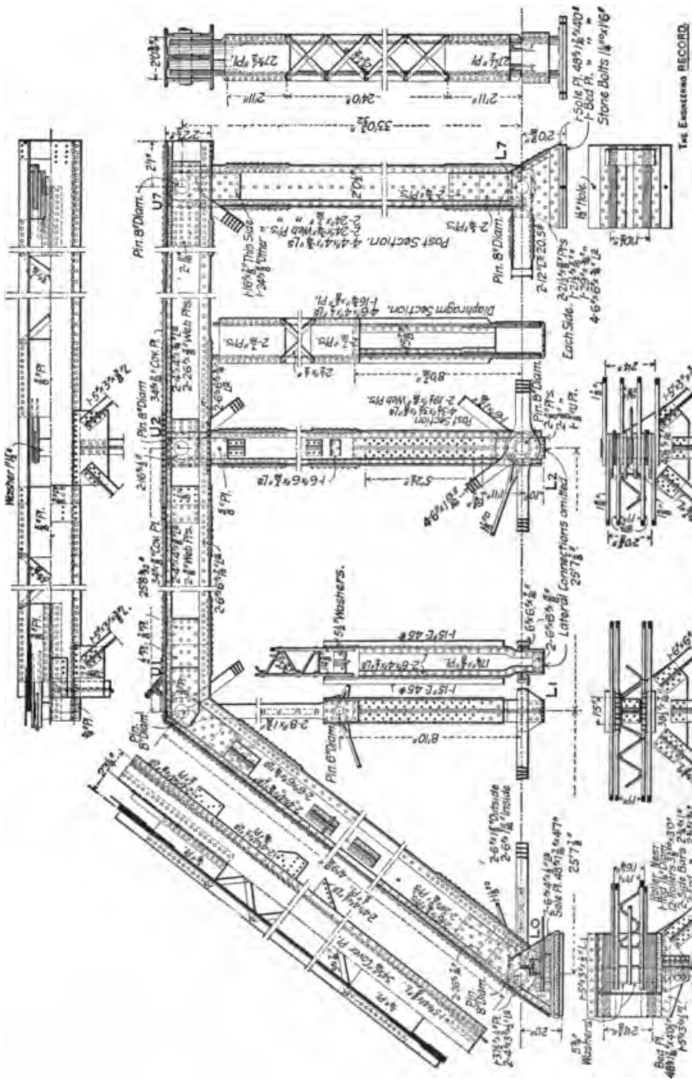


Fig. 111. Union Railroad Double-track Through Bridge over the Pittsburg, Virginia and Charleston Railroad at Port Perry, Pa. Span, 177¼ feet. Built in 1897. See references to various details in Arts. 73, 74, 77, 79, 81, and 96

Span, 235′ 7″. Description and partial detail drawings of one of the spans of the double-track deck bridge over the Schuylkill river near Girard Avenue, Philadelphia. The illustrations are reproduced in Plate V and in Fig. 110. Many of the details are referred to in the preceding articles of this chapter. The structure was designed for heavy traffic under comparatively high speeds. (See Proceedings of the Engineers' Club of Philadelphia, vol. 14, page 302, Jan., 1898, for an account by JOSEPH T. RICHARDS of the operation of moving aside the old Whipple truss bridge and putting this new bridge in its place in two minutes and twenty-eight seconds, on Oct. 17, 1897.)

The International Bridge, Buffalo. Engineering Record, vol. 43, page 566, June 15, 1901.

Description of the characteristic details of this single-track through bridge with some of their dimensions for the trusses whose span is 244′ 7″. The only details shown in the drawings are splices in the upper and lower chords, the connection of the floor beam with the stiff lower chord, and one of the segmental rollers.

The Victoria Jubilee Bridge at Montreal; Grand Trunk Railway. Engineering News, vol. 38, page 130, Aug. 26, 1897.

The small-scale elevation, section, and plan give the stresses in the members of the Pratt truss whose span is 253′ 11½″, and the composition of the truss members and of the lateral and sway bracing. The cantilever ends of the floor beams of this double-track through bridge support a roadway and sidewalk on each side.

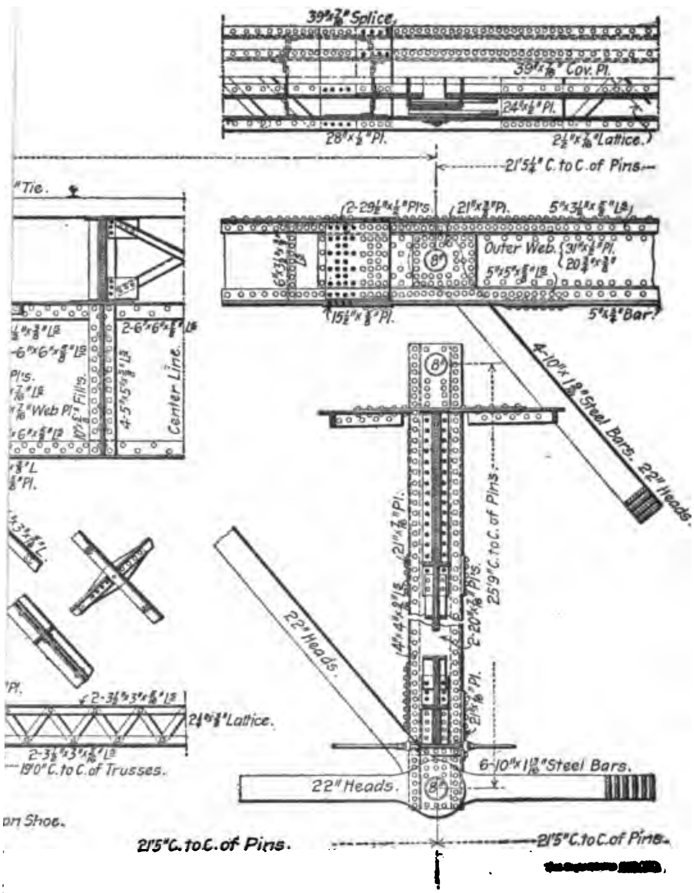
The Reconstruction of the Glasgow Bridge on the Chicago and Alton Railway. By W. D. TAYLOR. Engineering Record, vol. 43, page 241, March 16, 1901.

Description and detached drawings of several characteristic details of the through Pratt truss with curved upper chord whose span is 339¼′, and of the deck Pratt truss whose span is 139′ 5⅛″. The deck truss is supported at the top chord. The bridge is a single-track structure. Views of the completed bridge and of the falsework are given.

The Victoria Bridge at Montreal. Engineering Record, vol. 38, page 488, Nov. 5, 1898.

Span, 347′ 11½″. The composition of the truss members of the lateral and sway bracing and of the floor are marked on the small-scale drawings of a part of the Pettit truss. Similar data are also given for a part of the smaller Pratt truss spans. (See one of the preceding references.)

Railroad.



BLE-TRACK DECK BRIDGE OVER THE SCHUYLKILL
GIRARD AVENUE, PHILADELPHIA.

EET 7 INCHES. ERECTED IN 1897.

e details in Arts. 71, 73, 74, 77, 78, 80, and 81.

The Davenport and Rock Island Bridge over the Mississippi River. Engineering News, vol. 43, page 26, Jan. 11, 1900.

Span, 361 feet. The general character of the single-track through Pettit truss bridge is shown by a very small scale drawing, on which are marked the composition of most of the tension members. The article relates principally to the swing span.

Special Bridge and Viaduct Construction in Western Pennsylvania. Engineering Record, vol. 41, page 516, June 12, 1900.

Span, 396′ 8″. Double-track through Pettit truss bridge of the Union Railroad over the Monongahela river at Port Perry, Pa. Built in 1897. Description and partial detail drawings of the heavy truss, showing its most important features. Provision was made for adding a third truss later. One of the tracks is a hot-metal route with special fireproof protection.

The Rankin Bridge. Engineering Record, vol. 44, page 465, Nov. 16, 1901.

Span, 495′ 8¾″. Double-track through Pettit truss bridge of the Union Railroad over the Monongahela river at Rankin, Pa. Built in 1900. Description and partial illustration of details. One of the tracks is a hot-metal route. Both of these bridges of the Union Railroad were designed for the heaviest live load, consisting of two 192½-ton locomotives followed by a uniform load of 5000 pounds per foot per track. The total weight of the steel work alone in one of the long spans of the Rankin bridge is about 2800 tons. (See Fig. 108.)

Superstructure of the Delaware River Bridge at Bridesburg, Philadelphia, for the Pennsylvania and New Jersey Railroad Company. By PAUL L. WÖLFEL. Proceedings of Engineers' Club of Philadelphia, vol. 14, page 154, 1897.

Span, 533 feet. Description of several special features in the design of this double-track through bridge with subdivided panels and curved upper chord. Built in 1896.

The Delaware River Bridge at Bridesburg. Engineering Record, vol. 40, page 594, Nov. 25, 1899.

Description of the principal details. Two of the fine views relate to the fixed spans. Some of this description was reprinted from Mr. WÖLFEL'S paper.

Die Brücke der Pennsylvania-Eisenbahn über den Delaware bei Philadelphia. Von F. C. KUNZ. Allgemeine Bauzeitung, Wien, Heft 1, 1901.

Description of the design, construction, and erection, illustrated by numerous views and a number of large plates showing many of the details of the fixed and swing spans. Analyses of their weights are included. (See Figs. 10 and 107.)

CHAPTER IX.

DESIGN OF A PIN TRUSS BRIDGE.

ART. 83. SPECIFICATIONS.

Let it be required to design a single-track through railroad bridge whose trusses are of the Pratt type and whose span is 175 feet between centers of end pins. The cross-ties, foot planks, and guard timbers are to be of long-leaf Southern yellow pine, the truss pins of medium steel, and the rest of the structure of soft rolled steel.

The trusses are to be spaced 17 feet center to center and the clear opening is not to be less than that shown in Fig. 112.

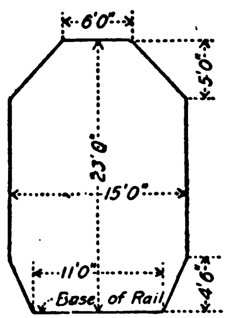


Fig. 112.

LOADS.

The live load is to be class Q of WADDELL'S compromise standard system (Art. 32), but the equivalent live loads given on the diagrams in WADDELL'S specifications are to be used instead of the actual wheel concentrations.

The effect of impact and vibration shall be added to the maximum stresses resulting from the above live load, and is to be determined by the following formula:

$$I = S\left(\frac{300}{L + 300}\right),$$

in which I is the impact, S the computed maximum live-load stress, and L the length of the loaded distance in feet which produces the maximum stress in the member.

To provide for wind stresses due to the pressure of the wind on the truss and train, as well as for lateral vibrations from high-speed trains, the wind load

on the lower lateral system of through bridges shall be 600 pounds per linear foot, 450 pounds of this to be treated as a moving load, and as acting on a train of cars at a line 6 feet above base of rail; and the static wind load on the upper lateral system shall be 150 pounds per linear foot.

The total traction load on any portion of the bridge is to be taken as 20 percent of the greatest live load that can be placed on that portion. No percentage of impact is to be added to traction loads.

UNIT STRESSES.

No metal less than three-eighths of an inch in thickness shall be used except for filling plates.

All parts of the structure shall be so proportioned that the sum of the maximum loads, together with the impact, shall not cause the tensile stress to exceed 15 000 pounds per square inch. The same limiting unit stresses shall also be used for members stressed by wind pressure, or momentum of moving train. Net sections must be used in all cases in calculating tension members, and, in deducting rivet holes, they must be taken one-eighth of an inch larger than the size of the rivets. The net section of any tension flange or member shall be determined by a plane cutting the flange square across at any point. The greatest number of rivet holes which can be cut by any such plane, or whose centers come nearer than two and a half inches to said plane, are to be deducted from the gross section when computing the net area.

For compression members, this permissible stress of 15 000 pounds shall be reduced in proportion to the ratio of the length to the least radius of gyration of the section by the following formula:

$$p = \frac{15\,000}{1 + \frac{1}{13\,500} \cdot \frac{l^2}{r^2}},$$

in which p is the permissible working stress per square inch in compression, l the length of piece in inches, between centers of connections, and r the least radius of gyration of the section in inches. No compression member, however, shall have a length exceeding 100 times its least radius of gyration, except those members of trusses whose main function is to resist tension and all compression members for wind bracing which may have a length not exceeding 120 times the least radius of gyration.

Members subject to alternate stresses of tension and compression in immediate succession (as counter stresses in web members of trusses) shall be so proportioned that the total sectional area is equal to the sum of the areas required for each stress.

In case the maximum stresses due to wind added to the maximum stresses due to vertical loading (including impact) shall exceed 19 000 pounds per square inch, properly reduced for compression, addition must be made to such sections until this limit is not exceeded. The permissible stresses for the connections shall be increased proportionately. Should the stresses be reversed in any possible case, proper provision must be made for such stresses in the opposite direction.

To insure the stability of bridges under increased live loads, a live load shall be assumed 100 per cent greater than that previously provided for in this specification. If the resultant stress per square inch in any member is more than twice the permissible stress previously specified, additions must be made to the sections until that limit is not exceeded.

The shearing stress on rivets, bolts, or pins shall not exceed 11 000 pounds per square inch of section; and the pressure upon the bearing surface of the projected semi-intrados (diameter times thickness) of the rivet, bolt, or pin hole shall not exceed 22 000 pounds per square inch. In field riveting, the number of rivets thus found shall be increased 25 percent if driven by hand, and 10 percent if satisfactory power riveters are used. The amount of field riveting must be reduced to a minimum, without, however, diminishing the number of rivets requisite for strength and rigidity. Whenever it is practicable, all designs are to be so made that the field rivets can be driven readily. For members of any importance, more than two rivets are to be used for each connection. Rivets are not to be used in direct tension.

If the extreme fiber stress resulting from the bending due to the weight only of any member does not exceed 10 percent of the specified unit stress, the effect of such bending may be ignored; but if it does so exceed, its effect must be combined with those of the other stresses, using, however, for determining the sectional area, a unit stress 10 percent greater than that specified.

In general, all trusses shall have main end posts inclined. The effective length of pin-connected spans shall be the distance between centers of end pins of trusses. The effective depth shall be the perpendicular distance between gravity lines of chords, which lines must pass through the centers of pins.

GENERAL PRINCIPLES OF DESIGNING.

[From WADDELL'S Specifications.]

The axes of all members of trusses or girders, and those of lateral systems coming together at an apex of a truss or girder, must intersect at a point whenever such an arrangement is practicable; otherwise the greatest care must be employed to insure that all the induced stresses and bending moments caused by the eccentricity be properly provided for.

Truss members and portions of truss members must always be arranged in pairs symmetrically about the central plane of the truss, except in the case of single members the axes of which lie in said central plane of truss.

In proportioning main members of bridges, symmetry of section about two principal planes at right angles to each other is to be attained wherever practicable; but in designing top chords and inclined end posts, this rule cannot be followed.

In both tension and compression members the center line of applied stress must invariably coincide with the axial right line passing through the centers of gravity of all cross-sections of the member taken at right angles thereto.

The principle of symmetry in designing must be carried even into the riveting; and groups of rivets must be made to balance about center lines and central planes to as great an extent as is practicable.

In all structural metal work, excepting only the machinery for operating movable bridges, no torsion on any member shall be permitted if it can possibly be avoided; otherwise the greatest care must be taken to provide ample strength and rigidity for every portion of the structure affected by such torsion.

In all main members having an excess of section above that called for by the greatest combination of stresses, the entire detailing is to be proportioned to correspond with the utmost working capacity of the member, and not merely for the greatest total stress to which it may be subjected. In this connection, though, the reduced capacity of single angles connected by one leg only must not be forgotten.

Designs must invariably be made so that all metal work after erection shall be accessible to the paint brush, excepting, of course, those surfaces which are in contact with each other or with the masonry. This requirement rules out all closed columns of every type and description.

In general, details must always be proportioned to resist every direct and indirect stress that may ever come upon them under any probable circumstances, without subjecting any portion of their material to a stress greater than the legitimate corresponding working stress.

In all designs simplicity in both main members and details is to be considered of the greatest importance. In all structures rigidity is to be considered quite as important an element as mere strength.

In all designing true economy must be given the utmost consideration; and no useless material must be employed, every pound of metal in the structure having a legitimate function; but economy of material must not be quoted as an excuse for using inferior details or scamping the work in respect to strength. rigidity, or appearance.

In all structural work the subject of æsthetics must be duly considered; and all designs are to be made in harmony with the principles thereof, to as great an extent as the money available for the work will permit, or as the environment of the structure calls for.

For convenience of reference the remaining items of the specifications are printed in the following articles to which they relate. In the extracts from specifications which are printed in this chapter, slight modifications are sometimes made for the sake of simplifying the problem for the student who is taking his first lessons in designing, rather than to express disagreement with the original provisions.

Seven panels will be adopted for the trusses, making the panel length 25 feet. Six panels would make the panel length over 29 feet, thus giving a comparatively heavy floor system; and since the spacing of the trusses is 17 feet, it would give poorer proportions to the lateral systems, and therefore render them less effective. On the other hand, eight panels make the panel length less than 22 feet, which is rather short for a bridge of this span.

ART. 84. FLOOR TIMBERS.

SPECIFICATIONS. — Cross-ties, foot planks, and guard timbers shall be of long-leaf Southern yellow pine. The wooden floor shall be so designed as to insure safety from passing trains for the railroad employees. The spaces between cross-ties shall, in general, not be less than five inches nor more than six inches wide. The sizes of the cross-ties shall be such as to give the requisite resistance to bending, under the assumption that the load on one pair of wheels is distributed equally over three ties, the effect of impact being considered. No cross-tie shall be less than seven, or preferably, eight inches wide, nor less than six inches deep, nor less than 10 feet long.

Cross-ties shall be notched not less than one-half inch over the stringers and be given a full and even bearing on the flanges; and each alternate cross-tie shall be secured thereto at each end by a $\frac{3}{4}$-inch hook bolt, having at the hook end a square shank, at least two inches long, to prevent the bolt from turning. All timber bolts shall be of soft steel with cold-pressed threads.

Outer guard timbers shall be 6″ × 8″, laid flat, notched one inch over the cross-ties, and placed so that their inner faces shall be just twelve inches from the gage planes of the rail. Each guard timber must be bolted to each alter-

nate cross-tie by a ¾-inch screw bolt, the head of which shall be countersunk into the wood by means of a cup-shaped washer. Each guard timber must be spliced over a cross-tie with a half-and-half joint of at least six inches lap, through which must pass a ¾-inch screw bolt. Guard timbers shall extend over all piers and abutments.

Inner guard rails shall consist of steel track rails securely fastened to the cross-ties, so that the outer sides of the heads shall be just five inches from the gage planes of the track rails.

The allowable tension in the extreme fibers of long-leaf Southern yellow pine timber in bending, the effect of impact being considered, shall be 2000 pounds per square inch.

In estimating the dead load the weight of yellow pine shall be assumed at 3¾ pounds per foot, board measure, and that of the rails, spikes, and joints at 160 pounds per linear foot of track.

The greatest stress in the cross-ties is produced by the alternative loading specified. (See Arts. 32 and 83.) The weight on one axle is 58 000 pounds. The impact is also 58 000 pounds. If the cross-ties be 8 inches wide and spaced 6 inches in the clear, three ties and spaces will cover a length of 3½ feet. Assuming the total weight of the track as 450 pounds per linear foot, the weight for a length of 3½ feet is 1575 pounds. The total load on three ties is therefore 117 575 pounds, and for each rail on one tie 19 600 pounds. The dead load is relatively so small that it may be assumed to be also concentrated at the track rails, without appreciable error. The stringers are spaced 6½ feet apart (Art. 85). The cross-tie is a beam with two concentrated loads, each of 19 600 pounds, spaced 4 feet 11½ inches apart and placed symmetrically with respect to the supports furnished by the stringers. The bending moment is therefore 181 300 pound-inches. For a unit stress of 2000 pounds per square inch and a width of 8 inches, the required depth of the cross-tie is found to be 8.24 inches. A depth of 9 inches will accordingly be taken. If the width were 7 inches, the depth would also be 9 inches, but this width makes the compression under the rail rather high

Let the cross-ties be alternately 10 and 14 feet in length, the additional length being required to support a 2-inch plank footwalk on each side of the track. A computation of the weight of the track shows that it equals about 440 pounds per linear foot.

ART. 85. TRACK STRINGERS.

SPECIFICATIONS. — The stringers shall be spaced $6\frac{1}{2}$ feet between centers. Stringers for truss bridges shall invariably be built of plates and angles, and no cover plates will be allowed for the flanges. Their depths shall be made not less than the most economic ones in respect to weight of metal required, provided that the bridge clearance will permit, and never less than one-twelfth of the span. The stringers are to be riveted to the webs of the floor beams. No splices will be allowed in their flanges, nor any in their webs, provided that sufficiently long web plates are procurable. The compression flanges shall be made of the same gross section as the tension flanges; and they shall be so stiffened that the unsupported length shall never exceed twelve times the width of flange.

Rigid diagonal bracing of angles is invariably to be used between the top flanges of stringers, and rigid cross-frames are to be employed near all expansion points. If the panel length exceed thirty feet, there shall be a cross-frame at mid-length between the contiguous stringers of each track; but for all shorter panels the rigid lower lateral diagonals which are riveted to the bottom flanges will stiffen the latter sufficiently.

The effective length of the stringers shall be the distance between centers of floor-beam webs. The effective depth shall be the distance between the lines passing through the centers of gravity of the sections of the upper and lower flanges. The unit stress in the net section of the tension flange shall not exceed 15 000 pounds per square inch. The web shall be regarded as resisting its proportion of the bending moment. The shearing stress in the web plate shall not exceed 10 000 pounds per square inch. See other unit stresses in Art. 83. The web shall have stiffeners riveted on both sides, at intervals not exceeding the full depth of the web plate when its thickness is less than one-sixtieth of the unsupported distance between the flange angles. The end stiffeners are to be faced or otherwise treated so as to make the stringers of exact length throughout, and so as to effect a uniform bearing of the end stiffeners against the webs of the floor beams. The general rules for riveting applied to the design of plate girders (Chap. VII) are to apply also to stringers. Flanges of stringers carrying the vertical load from the cross-ties shall have their rivets spaced uniformly from end to end, and at the minimum distance employed.

Since the design of a plate girder is explained in detail in Chapter VII, and a stringer is a plate girder of short span and simple form, only the principal results of the computations are given in this article, with but very brief descriptions of the methods employed.

The span of the stringer equals the panel length of the truss, or 25 feet. The equivalent uniform live load for a span of 25 feet is 9850 pounds per linear foot per track, and the maximum bending moment for one stringer is 384 800 pound-feet. The coefficient of impact for the same span is 0.923, and the corresponding moment, 355 100 pound-feet. The weight of the track carried by one stringer is $220 \times 25 = 5500$ pounds, while the weight of one stringer and of half the lateral bracing is assumed to be 5100 pounds, making the dead load 10 600 pounds. The dead-load moment is 33 100, and the total bending moment 773 000 pound-feet.

The diagram of end shears gives 146 000 pounds for a span of 25 feet, making the vertical live-load shear at the end of each stringer 73 000 pounds. The allowance for impact is 67 380 pounds, and the shear due to dead load 5300 pounds, thus giving a total vertical shear of 145 680 pounds.

Since the specifications require the depth to be taken large enough to make the weight of metal a minimum, let the depth of the web plate be taken as 42 inches, or a little less than one-seventh of the span. The method of determining this value, as well as the approximate weight of the stringer, will be explained later in this article. Let the web project one-half inch above the upper flange angles, so that the cross-ties need to be notched only over the web plate and not over the full width of the flange.

For the specified shearing stress of 10 000 pounds per square inch the net section of the web must be 14.57 square inches.

A thickness of $\frac{7}{16}''$ allows for 9 rivet holes of $\frac{15}{16}''$ diameter, or only 8 of $1''$ diameter. This is probably just about sufficient for the net section, which is that through the inner line of rivets in the end connection of the stringer. This thickness, however, requires stiffeners to be used, which may be avoided by increasing the thickness to $\frac{1}{2}''$. As flange angles 6 inches wide are most suitable for stringers without cover plates, the clear distance between flange angles is about $29\frac{1}{2}$ inches. It will be economical to do this, since seven pairs of stiffener angles, $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$, weigh about 400 pounds, while the increased weight of the web plate is only about 220 pounds. The extra material in the web plate also increases the stiffness of the stringer.

The outer row of rivets in the 6-inch flange angles is $2\frac{1}{4}$ inches from the backs of the angles (Art. 34). According to the method explained in Art. 56, a section of the web plate passing through the outer rivets of the flanges has its resisting moment reduced to 88.4 percent of that of the solid plate, and hence one-sixth of this, or 14.7 percent of its gross section, may be used as equivalent flange area. This gives $0.147 \times 41.5 \times \frac{1}{2} = 3.06$ square inches, provided the half inch which the web plate projects above the top flange is neglected. If the section be taken through the inner rivets, which are $4\frac{3}{4}$ inches from the backs of the angles, the corresponding values are 91.4 percent, 15.2 percent, and 3.15 square inches. It will be observed that the ratios for the equivalent flange areas are a little over one-seventh.

For the specified tensile stress of 15 000 pounds per square inch and an estimated effective depth of 38.2 inches for the section through the outer rivet holes and the back of the lower flange angles placed $\frac{1}{8}''$ below the edge of the web, the required net area of the lower flange is

$$\frac{773\,000 \times 12}{38.2 \times 15\,000} = 16.18 \text{ square inches,}$$

and that of the flange angles alone is $16.18 - 3.06 = 13.12$ square inches. Two angles $6'' \times 6'' \times \frac{5}{8}''$ give a net area of $2(7.11 - 0.625) = 12.97$ square inches. If the section be taken through the inner rivet holes, the effective depth is 38.45 inches, and the required net area of the flange angles is

$$16.08 - 3.15 = 12.93 \text{ square inches.}$$

In order to avoid using angles $\frac{11}{16}''$ thick let the backs of the lower flange angles be placed $\frac{1}{4}''$ instead of $\frac{1}{8}''$ below the edge of the web plate. In this case the smaller effective depth becomes $41.5 + \frac{1}{4} - 1.73 - 1.68 = 38.34$ inches. This reduces the required net area from 13.12 to 12.97 square inches, and hence the $6'' \times 6'' \times \frac{5}{8}''$ angles may be accepted.

Let the rivet pitch in the flanges be determined next. The maximum vertical shear at the end is 145 680 pounds, and the increment of flange stress per linear inch (see Art. 59) is

$$\frac{12.97}{16.03} \times \frac{145\,680}{38.34} = 3075 \text{ pounds.}$$

The vertical load on the flange, including impact, is

$$52\,790/42 = 1257 \text{ pounds.}$$

The resultant of these horizontal and vertical components is 3075 pounds. The allowable bearing value of a $\frac{7}{8}''$ rivet in a $\frac{1}{2}''$ web plate is 9630 pounds, and hence the theoretic rivet pitch at the end is $9630/3075 = 3.13$ inches. In accordance with the specifications a uniform pitch of 3 inches will be used throughout.

Since the vertical angles which connect the end of the stringer to the web of the floor beam are to be straight, fillers whose thickness equals that of the flange angles are placed

under the connecting angles and made wide enough to receive an extra row of rivets beyond the angles. The value of a $\frac{7}{8}''$ rivet in single shear at 11 000 pounds per square inch is 6610 pounds, while the bearing value is 9630 pounds, as stated in the preceding paragraph. As the rivets connecting the angles and the fillers to the web plate are in double shear, the bearing value of the rivets will govern in determining the number required to transmit the shear from the web plate into the connections. This number is therefore $145\,680/9630 = 16$.

In finding the minimum number of rivets which must pass through the angles, the value of the rivets in double shear will govern, since that is less than their bearing in the two angles combined, or in the web and filler plates combined. The number required must therefore not be less than $145\,680/132\,220 = 11$. Those passing also through the flange angles cannot be counted in either number, since their duty is to transmit the flange stresses.

The rivets connecting the other legs of these angles to the web of the floor beam are field rivets, and since there are two rows of rivets in single shear, the number in each angle must be one-fourth greater than 11, which gives 14. While this number of rivets may be crowded into a single row, it makes the pitch about $2\frac{5}{8}$ inches, and it is therefore preferable to use $6'' \times 6''$ angles. By counting the rivets in the flange angles, the number in the other leg of each angle is 15, and as the rivets in the adjacent rows must stagger, it is necessary to put 15 rivets into the other leg also. The thickness of the connecting angles cannot be determined theoretically; but experience shows that $\frac{7}{16}''$ will be sufficient, although $\frac{1}{2}''$ is frequently employed. As the web is $\frac{1}{2}''$ thick and the stringer has to be faced at the ends, the angles will be made $\frac{1}{2}''$ thick. The filler plates must then be $9'' \times \frac{5}{8}''$. Fig. 113 shows that

all the necessary conditions are satisfied by placing 6 rivets in the fillers outside of the angles.

Sometimes fillers are used in practice whose width does not exceed that of the angles; but such an arrangement is not to be approved. If in the above example 16 rivets could be put through the connecting angles in the space between the

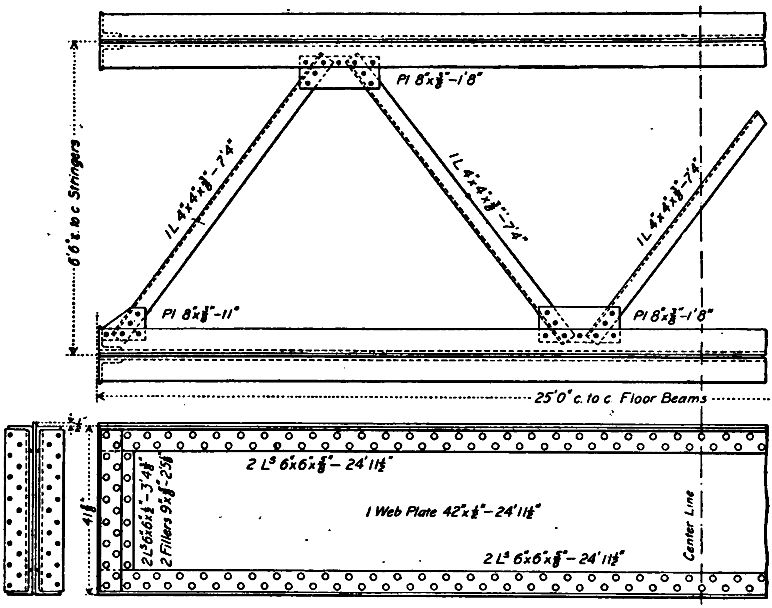


Fig. 113.

flange angles without reducing the pitch below the minimum allowed, the connection would not have the same degree of strength; for some tendency to bend the rivets is developed in transferring the entire shear directly from the web to the angles, to resist which extra rivets would be required. With the arrangement shown in Fig. 113 extra rivets are often necessary when only a single row of rivets is placed in the angles in order to keep the pitch from exceeding the maximum allowed in the row outside of the angles.

The skeleton diagram of the lateral system is given in Fig. 114. According to the specifications the upper flange must be stayed laterally at intervals not exceeding 12.56 feet, and this condition is satisfied by the arrangement shown. It also makes the connections with both girders exactly alike, so that the same patterns may be used for both. The stresses S_1 and S_2, due to the wind load, are found to be ± 4970 and ± 3830 pounds, respectively. The length of a lateral is 90 inches, and hence the least radius of gyration must not be less than $90/120=0.75$. This requires the use of $4'' \times 4''$ angles, and with a minimum thickness of $\frac{3}{8}''$ the area is considerably in excess of that required for the stresses, including those due to eccentric connections.

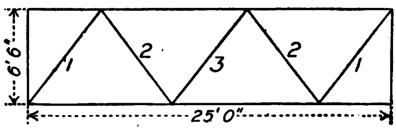


Fig. 114.

The final estimate of the weight of the stringer is now computed with the aid of the tables in a handbook.

	Pounds.
4 flange angles, $6'' \times 6'' \times \frac{5}{8}'' \times 24' \, 11\frac{1}{2}''$, @ 24.2 lbs.	2416
1 web plate, $42'' \times \frac{1}{2}'' \times 24' \, 11\frac{1}{2}''$, @ 71.4 lbs.	1782
4 connecting angles, $6'' \times 6'' \times \frac{1}{2}'' \times 3' \, 4\frac{3}{8}''$, @ 19.6 lbs.	264
4 fillers, $9 \times \frac{5}{8}'' \times 2' \, 5\frac{1}{8}''$, @ 19.13 lbs.	186
Half lateral system:	
$2\frac{1}{2}$ lateral angles, $4'' \times 4'' \times \frac{3}{8}'' \times 7' \, 4''$, @ 9.8 lbs.	180
2 connecting plates, $8'' \times \frac{3}{8}'' \times 1' \, 8''$, @ 10.2 lbs.	34
1 connecting plate, $8'' \times \frac{3}{8}'' \times 11''$ (corner clipped)	8
238 pairs of rivet heads, @ 0.369 lb.	88
Total	4958

In addition to their own weight and that of their lateral system, the stringers support a part of the weight of the lower laterals of the bridge and of their connections to the stringers. The assumed weight of 5100 pounds exceeds that just obtained by a sufficient amount to avoid the necessity of revising the moments and shears due to the dead load. It should also be

remembered that the end connections are really supported by the floor beam, and their weight therefore causes no stresses in the stringers.

The weight of one stringer and one-half of the lateral system is distributed as follows:

	Weight in Pounds.	Percentage of Total Weight.	
Flanges	2416	48.7	84.6
Web plate	1782	35.9	
End connections with floor beams	450	9.1	15.4
Half lateral system	222	4.5	
Rivet heads	88	1.8	
	4958	100.0	100.0

This table also shows that the specification in regard to the depth of the stringer is practically satisfied, for the weight of the flanges is only 184 pounds greater than that of the web, including the end connections. The percentages of these two items are 48.7 and 45 respectively. This indicates that theoretically the total weight is near the minimum. As it seemed probable that a depth of 44 inches might give better results, another design was made which gave the following weights: Flanges, 2187 pounds; web plate, 2098 pounds; end connections, 459 pounds; rivet heads, 78 pounds; lateral bracing, the same as before; and the total, 5055 pounds. The web had to be increased in thickness to $\frac{9}{16}''$ in order to avoid the use of stiffeners, and the flange angles were reduced in thickness to $\frac{9}{16}''$. In this case the weight of the flanges is less than that of the web, and hence the depth of 42″ gives practically the minimum material.

The above analysis of the weight of a stringer is also useful in checking the assumed weight after but few preliminary com-

putations are made to determine the web and flange section. In an example published in the first edition of this part of the text-book, in which flange cover plates and web stiffeners were employed, the live load being only about three-fifths as heavy, and the economic depth 38 inches, the combined weight of the flanges and web plate was found to be 77.6 percent of the entire weight. This indicates that even in extreme cases the variation in this percentage is not very large.

ART. 86. FLOOR BEAMS.

SPECIFICATION. — The effective length shall be the perpendicular distance between the central planes of trusses. All spans shall have end floor beams riveted rigidly to the trusses, to support the stringers. The intermediate floor beams in through bridges are to be riveted between the posts. For unit stresses see Art. 83. Most of the specifications relating to stringers apply also to floor beams.

For convenience in erection, bracket angles are riveted to the lower flange of the floor beam or to the web just above the flange, on which to support the stringers until their end-connecting angles are riveted to the floor-beam webs.

Assuming that the vertical legs of the flange angles do not exceed 4 inches, it is found that a depth of 54 inches will bring the top of the cross-ties between 2 and 3 inches higher than the top of the floor beam, provided the bracket angle has a 5-inch vertical leg and is riveted to the lower flange angles.

The floor beam carries, in addition to its own weight, two concentrated loads 3′ 3″ from its center, each load consisting of the maximum sum of the adjacent reactions of the stringers on both sides. This sum includes the weight of one stringer, and of the track which it supports, and the corresponding live load. Using the same values as in the preceding article, the dead load of one stringer is $5500 + 5100 = 10\,600$ pounds. The equivalent uniform live load must be taken for a span of two panel lengths, or 50 feet, and by means of WADDELL's diagram it is found to be

8000 pounds per linear foot per track. The live load at each stringer connection is therefore $4000 \times 25 = 100\,000$ pounds. The allowance for impact is 85 700 pounds, and the total load 196 300 pounds. The approximate weight of the floor beam is assumed to be 4200 pounds.

The maximum vertical shear is 198 400 pounds, and for the allowable shearing stress of 10 000 pounds per square inch, the net section of the web is 19.84 square inches. A thickness of $\frac{1}{2}''$ will allow for about 14 rivet holes in the net section of the splice, which is probably sufficient. Assuming one-eighth of the gross section as the equivalent flange area of the web plate, its value is 3.37 square inches.

It is desirable to determine the rivet pitch approximately before the flange angles are definitely selected, in order to know whether the rivets can be placed in a single row. The effective span of the floor beam is 17 feet, the distance between the centers of trusses. The bending moment due to the two concentrated loads is 1 030 600 pound-feet, and that due to the weight of the floor beam is 8900 pound-feet, making the total 1 039 500 pound-feet. The moment diagram is shown in Fig. 115, the moment due to dead load being laid off below the axis. Assuming an effective depth of 53 inches, and using the specified unit stress, the required net area of the lower flange is

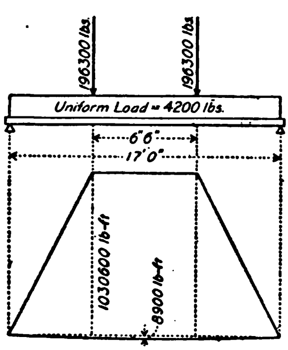


Fig. 115.

$$\frac{1\,039\,500 \times 12}{53 \times 15\,000} - 3.37 = 12.32 \text{ square inches.}$$

The approximate increment of flange stress per linear inch between the end and the stringer connection is $12.32 \times 15\,000/63$

$= 2934$ pounds. The pitch is, therefore, about $9630/2934 = 3.28$ inches, the bearing of a $\frac{7}{8}''$ rivet in the $\frac{1}{2}''$ web plate being 9630 pounds. This indicates that only one row of rivets is needed to connect the flange angles to the web.

Let the following composition of the flange be taken, which furnishes a net area of 12.30 square inches:

2 angles, $5'' \times 4'' \times \frac{9}{16}''$,	$2\ (4.75 - 1.13) =$	7.24	square inches.
1 cover plate, $11'' \times \frac{9}{16}''$,	$(6.19 - 1.13) =$	5.06	
		12.30	

The center of gravity of the solid section of the upper flange is $0.55''$ below the backs of the angles, and that of the net section of the lower flange is $0.60''$ above the backs of the angles, the rivet holes in the vertical legs of the angles being also deducted, since they are less than $2\frac{1}{2}$ inches from the section through the rivet holes in the horizontal legs of the angles and in the cover plates. Placing the backs of the angles $\frac{1}{8}''$ beyond the edges of the web plate, the effective depth is $54 + 0.25 - 0.60 - 0.55 = 53.1$ inches. This value reduces the required net area of the flanges to $15.66 - 3.37 = 12.29$ square inches. The above composition of the flanges may therefore be adopted, provided it is found later that the equivalent flange area of the web is not less than that assumed.

As shown in Fig. 116, the web plate will be spliced in order to allow the bottom of the floor beam to be on the same level as that of the post, that the connecting plates of the lateral system may be riveted to both, and also to allow the end web plate to be extended above the upper flange for the connection to the post. The method of designing this splice is the same as that explained in Art. 56. The rivets in the lower half of the inner row of rivets are located at the following distances from the neutral surface, all expressed in inches: 0, 3.5, 6.5, 9.5, 12.5, 15.5, 18.5, 21.5, and 24.625. The last distance is that of the

rivet in the flange angles. The squares of these numbers are: 0, 12.25, 42.25, 90.25, 156.25, 240.25, 342.25, 462.25, and 606.37. Their sum is 1952.1 inches^2. The resisting moment of the lower half of the solid web plate is 1 822 500 pound-inches, and the moment deducted by the rivet holes in this row is $7500 \times 1952.1/27.125 = 539\,750$ pound-inches, 7500 pounds being the tensile strength deducted by one rivet hole at the distance of the outer fiber, or 27.125 inches, from the neutral surface. If the outer row of rivets had the same number of rivets as the inner one, the resisting moment of the net section would be

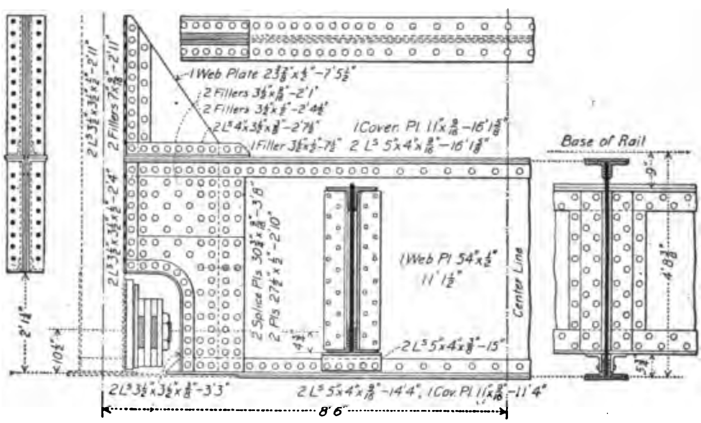


Fig. 116.

1 282 750 pound-inches. It must be a little larger than this because the net section of the web allows only 14 rivet holes. The sum of the moments of the bearing values of the rivets in the web plate is $9630 \times 1952.1/24.625 = 763\,400$ pound-inches, 9630 pounds being the bearing value of the rivet which is 24.625 inches from the neutral surface. This leaves a moment to be resisted by the rivets in the outer row of 519 350 pound-inches, and requires Σy^2 (see Art. 56) to be 1328 inches^2. Let the rivets be omitted whose distances are 3.5, 9.5, and 15.5 from the neutral surface, making Σy^2 equal to 1609.3 inches^2.

On revision the reduction of resisting moment due to the rivet holes in the outer row is $7500 \times 1609.3/27.125 = 445\,000$ pound-inches, making the net resisting moment of the web plate 1 377 500 pound-inches. The required moment of the bearing values of the rivets in the outer row is then 614 000 pound-inches and $zy^2 = 1570$ inches^2. This indicates that the strength of the splice is practically equal to that of the net section of the web plate, the rivets in the flange angles being included in both rows. That is allowable, because in this case the flanges have a large excess of section at the web splice, and the spliced plates extend clear to the left end of the beam. As the right-hand row contains only 11 rivets, the web has sufficient area to resist the shear.

The net section of the web plate is 1 377 500/1 822 500 = 0.756 or 75.6 percent of the strength of the gross section, and hence one-sixth of this, or 12.6 per cent of the gross sectional area of the web plate, equals its equivalent flange area. This gives 3.40 square inches, which agrees almost exactly with the assumption previously made.

The upper cover plate is extended to the post, it being slotted at the end to allow the projecting web plate to pass through. The lower cover is extended as far as the connecting plate for the lateral system permits. The rivet pitch may next be revised. The increment of flange stress per linear inch is

$$\frac{12.30}{15.70} \cdot \frac{198\,400}{53.1} = 2927 \text{ pounds,}$$

and the pitch is $9630/2927 = 3.29$ inches. It may therefore be made $3\frac{1}{4}''$ in the spaces outside of the stringers, but is preferably reduced to 3 inches. In the space between the stringers the pitch is made 6 inches, the maximum allowed.

In the design of the stringer it was found that 30 field rivets are required to connect the two end angles of each stringer to

the floor-beam web. It remains to determine how many are required to carry the whole load of 196 300 pounds at each pair of stringer connections. Since the rivets are in double shear, their bearing in the web of the floor beam will govern, and the number is 196 300/9630 = 21 shop rivets or 28 field rivets. The number previously found will therefore be used.

The equivalent flange area of the web which must be used in the final determination of the net flange area is that of a section taken through one of the rows of rivets in the stringer connection. For a section through the outer row in which the rivets are farther from the neutral surface, their distances being 3.5, 8.5, 13.5, 18.5, and 24$\frac{5}{8}$ inches respectively, the resisting moment is 81.6 percent of that of the solid plate, and hence its equivalent flange area is 13.6 percent of 27 square inches, or 3.67 square inches. The composition of the flange therefore requires no revision.

In the end connection of the floor beam, the number of shop rivets required to transmit the shear from the web into the fillers is 198 400/9630 = 21, and the number needed to carry it into the angles is 198 400/13 220 = 15. To connect the end angles to the post requires 198 400/6610 = 30 shop rivets, but as field rivets are used their number must be increased to 38.

The splice plates are extended to the end of the beam in order to act also as fillers under the end angles and to transfer the stresses to these angles in the most direct manner. An additional pair of plates is placed on the sides of the floor beam around the corner cut, in order to strengthen the web around the cut and also to serve as fillers for the curved flange angles.

The following table gives the weight of one floor beam:

	Pounds.
2 flange angles, $5'' \times 4'' \times \frac{9}{16}'' \times 16'\ 1\frac{5}{8}''$, @ 16.2 lbs.	523
2 flange angles, $5'' \times 4'' \times \frac{9}{16}'' \times 14'\ 4''$, @ 16.2 lbs.	464
1 cover plate, $11'' \times \frac{9}{16}'' \times 16'\ 1\frac{5}{8}''$, @ 21.02 lbs.	339

	Pounds.
1 cover plate, $11'' \times \frac{9}{16}'' \times 11'\ 4''$, @ 21.02 lbs.	238
1 web plate, $54'' \times \frac{1}{2}'' \times 11'\ 1\frac{1}{2}''$, @ 91.84 lbs.	1022
2 web plates, $23\frac{7}{8}'' \times \frac{1}{2}'' \times 7'\ 5\frac{1}{2}''$, @ 40.59 lbs. (less 223 lbs.) .	383
4 splice plates, $30\frac{3}{4}'' \times \frac{9}{16}'' \times 3'\ 8''$, @ 58.83 lbs. (less 226 lbs.) .	637
4 filler plates, $27'' \times \frac{1}{2}'' \times 2'\ 10''$, @ 45.92 lbs. (less 201 lbs.) .	316
4 connecting angles, $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{1}{2}'' \times 2'\ 4''$, @ 11.1 lbs. . .	104
4 connecting angles, $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{1}{2}'' \times 2'\ 11''$, @ 11.1 lbs. . .	130
4 fillers, $7'' \times \frac{9}{16}'' \times 2'\ 11''$, @ 13.39 lbs.	156
4 angles, $4'' \times 3\frac{1}{2}'' \times \frac{3}{8}'' \times 2'\ 7\frac{1}{2}''$, @ 9.1 lbs.	96
4 fillers, $3\frac{1}{2}'' \times \frac{9}{16}'' \times 2'\ 1''$, @ 6.7 lbs.	56
4 fillers, $3\frac{1}{2}'' \times \frac{1}{2}'' \times 2'\ 4\frac{1}{2}''$, @ 5.95 lbs.	56
2 fillers, $3\frac{1}{2}'' \times \frac{1}{2}'' \times 7\frac{1}{2}''$, @ 5.95 lbs.	7
4 angles, $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}'' \times 3'\ 3''$, @ 8.5 lbs.	111
4 bracket angles, $5'' \times 4'' \times \frac{3}{8}'' \times 1'\ 3''$, @ 11.0 lbs.	55
596 pairs of rivet heads @ 0.369 lb.	220
Total	4915

The weight is distributed as follows:

	Weight in Pounds.	Per-cent.
Flanges	1564	31.8
Web plates	1405	28.6
Splices, end connections, etc.	1726	35.1
Rivet heads	220	4.5
Total	4915	100.0

This analysis shows how much material is required in order to bring the bottom of the floor beam even with the bottom of the post and to clear the chord bars. As shown in Fig. 116, there are five plates riveted together to form the web around the corner cut. As so large a percentage of the weight of the floor beam is concentrated near the ends, no revision is needed on account of the difference between the assumed and the actual weights.

The design of the end floor beam is left as an exercise for the student. See Plate IV for an example showing the form and details of its end connections. The design of the end floor

beam cannot be completed until after that of the end posts to which it is connected.

ART. 87. STRESSES IN TRUSSES.

The following data and dimensions are tabulated below for convenient reference:

Span, center to center of end pins	175′ 0″
Depth, between centers of chords	31′ 0″
Width, between centers of trusses	17′ 0″
Number of panels	7
Panel length	25′ 0″
Length of end post, center to center of pins, 39.825′ =	39′ 9.9″

$$\theta = 38^\circ\ 53' \qquad \theta' = 55^\circ\ 47'$$

$$\tan\theta = 0.8065 \qquad \tan\theta' = 1.4706$$

$$\sec\theta = 1.2847 \qquad \sec\theta' = 1.7784$$

The angle which the diagonals of the truss make with the vertical is θ, and that of the diagonals of the lateral systems with the struts is θ'. The opposite truss has the same letters as the one in Fig. 117, except that the letters are primed.

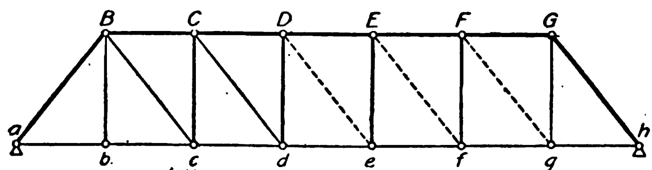


Fig. 117.

The weight per linear foot of the track is 440 pounds, that of the stringers and floor beams is 594 pounds, and that of the trusses and lateral systems is assumed to be 1336 pounds, making a total of 2400 pounds. The dead panel load per truss is 30 000 pounds; one-third of which will be applied at

the upper panel points, and the remainder at the lower panel points.

The equivalent uniform live load is shown by the diagram to be 6325 pounds per linear foot per track, which gives a panel load per truss of 79 100 pounds. The panel load for the suspender *Bb*, however, is equal to the floor beam reaction, or 100 000 pounds.

The specified wind load will be considered as equally divided between the windward and leeward panel points of each lateral system. The panel load for the upper system is 1875 pounds. The panel loads for the lower system are 1875 and 5625 pounds respectively for the static and moving wind loads. The panel load for the overturning moment of the wind pressure on the train is 7080 pounds, the distance from the base of the rail to the lower lateral system being estimated as 4.7 feet.

The stresses in the trusses and lateral bracing were computed according to the methods of Part I, and are given in the following tables, their values being expressed in kips, one kip being equal to 1000 pounds:

	END POST.	UPPER CHORD.		LOWER CHORD.		
	aB	*BC*	*DE*	*bc*	*cd*	*de*
Dead load	−115.6	−121.0	−145.2	+ 72.6	+121.0	+145.2
Live load	−304.8	−319.1	−382.8	+191.4	+319.1	+382.8
Impact allowance	−192.6	−201.6	−241.9	+121.0	+201.6	+241.9
Wind overturning:						
On truss, east	− 26.4	− 16.5	− 16.5	+ 16.5	+ 16.5	+ 16.5
On truss, west	+ 26.4	+ 16.5	+ 16.5	− 16.5	− 16.5	− 16.5
On train, east	− 27.3	− 28.6	− 34.3	+ 17.1	+ 28.6	+ 34.3
On train, west	+ 27.3	+ 28.6	+ 34.3	− 17.1	− 28.6	− 34.3
Wind on truss, east		0	+ 16.6	+ 16.5	+ 27.6	+ 33.1
Wind on truss, west		− 11.0	− 16.6	− 27.6	− 33.1	− 33.1
Wind on train, east				+ 49.6	+ 82.7	+ 99.3
Wind on train, west				− 82.7	− 99.3	− 99.3

	Diagonals.					Verticals.		
	Bc	*Cd*	*De*	*Ef*	*Fg*	*Bb*	*Cc*	*Dd*
Dead load	+ 77.1	+ 38.5	0	−38.5	−77.1	+ 20.0	− 40.0	−10.0
Live load	+217.8	+145.2	+87.1	+43.6	+14.5	+100.0	−113.0	−67.8
Impact allowance	+145.2	+102.5	+65.4	+34.8	+12.5	+ 85.7	− 79.8	−50.9
Wind overturning:								
On train, east	+ 19.5	+ 13.0	+ 7.8	+ 3.9	+ 1.3	+ 7.1	− 15.2	−10.1
On train, west	− 19.5	− 13.0	− 7.8	− 3.9	− 1.3	− 7.1	+ 15.2	+10.1

The stresses in *CD* are the same as those in *DE*, except for wind on truss, west, which is + 11.0 instead of + 16.6 kips. The stresses in *ab* are the same as those in *bc*, except those in the last four lines of the table, for which the following values are to be substituted: 0, − 16.5, 0, and − 49.6 kips.

The wind stresses in the upper laterals, expressed in kips, are: *BC'*, + 13.3; *CD'*, + 6.7; *DE'*, 0; while those in the struts are: *BB'*, − 3.8; *CC'*, − 5.6; *DD'*, − 1.9. The wind stresses due to both static and moving wind loads in the lower laterals are: *ab'*, + 80.0; *bc'*, + 56.2; *cd'*, + 35.3; *de'*, + 17.2; while those in the struts are: *aa'*, − 22.5; *bb'*, − 37.5; *cc'*, − 24.1; and *dd'*, − 12.13.

Art. 88. Sections of Intermediate Posts.

Specification. — The effective length shall be the greatest length between points of the axis that are rigidly held in the direction in which the strength is being considered. The least width of posts in pin-connected trusses shall be limited to 10 inches.

Let it be required to design the section of the post *Cc*. Neglecting the wind stresses, which are relatively too small to affect the result according to the specified unit stresses, the total stress to be considered is 232 800 pounds. A trial shows that 15-inch channels are required. Taking the radius of gyration about the neutral axis perpendicular to the web of the channels

as 5.43 inches, the value of l/r equals $31 \times 12/5.43 = 68.5$. By means of a table of unit stresses for values of l/r in the specified column formula ranging from 10 to 120, the corresponding average unit stress is found to be 11 110 pounds per square inch, which requires a sectional area of $232\,800/11\,110 = 20.96$ square inches. Referring to the handbook, it is found that two 15-inch channels, each weighing 40 pounds per linear foot, will give an area of 23.52 square inches. As the radius of gyration of these channels agrees with the value assumed, no revision is needed, and these channels are accordingly adopted. The flanges will be turned inward so as to avoid cutting them near the pin connections. It remains to determine the distance back to back of channels, in order that the moments of inertia about the two rectangular axes through the center of the section may be equal, thus rendering the column of equal strength against lateral flexure in these two directions, provided it may be equally free to bend in either direction. Let x be this distance, then by the application of the principles of moment of inertia (see Text-book on Mechanics of Materials, Arts. 23 and 51) there results the equation

$$2 \times 347.5 = 2\left[9.39 + 11.76\left(\tfrac{1}{2}x - 0.783\right)^2\right]$$

whose solution gives $x = 12.29$ inches. In this equation 347.5 inches^4 is the moment of inertia of one channel about the axis AB (Fig. 118), 9.39 inches^4 is the moment of inertia about the axis CD, and 0.783 inch is the distance from the axis CD to the back of the channel, all these elements being given in the handbook. Some of the handbooks also give the distances back to back of channels, in order to make the radii of gyration equal.

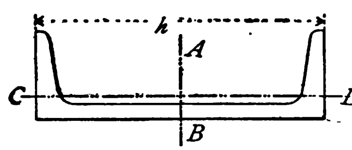


Fig. 118.

In a similar manner it is found that two 12-inch 25-pound channels are needed for the post *Dd*, the radius of gyration being 43 inches, the average unit compression 9850 pounds per square inch, the required sectional area 13.06 square inches, and the area furnished by the channels 14.70 square inches. The distance back to back of channels for equal radii of gyration is 10.07 inches. The channels in the posts are placed with their webs parallel to the plane of the truss.

Since all the post channels must have the same spacing, in order that the lengths of the floor beams may be the same; and as it is also important to make that distance as small as possible, because the width of the upper chord depends upon it, a computation may be made to see what the spacing of the channels in *Cc* must be, so that its area shall just equal that required. The value is found to be 10.31 inches. A uniform spacing of $10\frac{3}{8}$ inches will accordingly be adopted.

ART. 89. SECTIONS OF DIAGONALS AND SUSPENDER.

SPECIFICATION. — Counter-stresses must be provided for wherever caused by the increased live load (see Art. 83); and in case of reversal of stress the member must be designed to resist such reversal. The use of more than a single system of cancellation in bridges shall be confined entirely to lateral systems and sway bracing, except that in the middle panels of trusses two rigid diagonals connected at their intersection may, for appearance, be employed, provided that either diagonal have sufficient strength to carry the entire shear in either tension or compression, and that the adjacent vertical posts be figured accordingly. All through spans shall have stiff end vertical suspenders.

Since the minimum stress in *Bc* is a tension of 48 800 pounds, it may be composed of one or more pairs of eye-bars. The wind stress may be neglected in designing the member according to the specifications. For the unit tensile stress of 15 000 pounds per square inch, the sectional area must be 440 100/15 000 = 29.34 square inches. Two eye-bars, $8'' \times 1\frac{7}{8}''$, provide an area

of 30 square inches. For a depth of 7 inches the thickness would have to be $2\frac{1}{8}$ inches, which is not so desirable. The thickness of eye-bars ranges in practice from one-fourth to one-seventh of their depth or width. These limits are exceeded only in rare instances.

In accordance with the specifications no counter-tie is allowed in the third panel, and hence the diagonal *Cd* must be designed to take also a compression equal in magnitude to the tension given in the table for *Ef*, in Art. 87. Let two 15-inch 50-pound channels be tried. The required net area for tension is $286\,200/15\,000 = 19.08$ square inches. The radius of gyration $r = 5.23$ inches, the length $l = 477.9$ inches, $l/r = 91.4$, the allowable average unit compression $p = 9260$ pounds per square inch, and the required area for compression is $39\,900/9260 = 4.31$ square inches. The net section must therefore exceed 19.08 square inches, while the gross section must not be less than 23.39 square inches. The gross section of these channels is 29.42 square inches, their web thickness is 0.72 inch, and the grip of the rivets in their flanges is 0.625 inch. Except near the ends the only rivet holes are those in the flanges needed for the lacing. The end connections require pin plates to be riveted to the webs, and in channels of this size four lines of rivets will be used. As parts of the stay plates and pin plates will be opposite each other, it will be necessary to deduct the area of four rivet holes in the web and two in the flanges of each channel. This leaves a net section of $29.42 - 5.76 - 2.50 = 21.16$ square inches. The 45-pound channels will not do, as their net area falls below that required for the tension alone.

It remains to test the section by the provision of the specifications which is intended to make allowance for future increase of the live load. The area required for tension and compression under the increased live-load stresses are 17.80 and 6.54 square

inches respectively. The 50-pound channels may therefore be adopted.

According to the specifications each of the diagonals in the middle panel must be designed to take the entire stress in either tension or compression. The net area in tension is $152\,600/15\,000 = 10.17$ square inches. As the diagonals are riveted to common connecting plates at the middle, the length to be used in the column formula is one-half of the total length of *De*, or 239 inches. Let two pairs of angles, $5'' \times 3\frac{1}{2}'' \times \frac{9}{16}''$, laced together so as to form a section like Fig. 86 be tried. The lacing will be $\frac{3}{8}''$ thick, and hence the space between the backs of the angles must be $\frac{3}{4}''$. The radius of gyration $r = 2.60$ inches, $l/r = 91.9$, $p = 9210$ pounds per square inch, and the required gross area is $152\,600/9210 = 16.57$ square inches. The angles have a gross area of 17.88 square inches, and provide more than the needed net area when a rivet hole is deducted from both legs of the angles.

The suspender, *Bb*, will be designed as a stiff member for the reasons given in Art. 76, its composition being made like that of the intermediate posts. Its required net sectional area is $205\,700/15\,000 = 13.71$ square inches. Two 12 inch 30-pound channels will be selected, as they will furnish 13.80 square inches, after deducting two rivet holes in both the web and flanges of each channel. The net section is at the connection to the upper pin plates, and by staggering the rivets in the three rows connecting the webs to the pin plates it may be arranged that the flange rivets through the stay plates shall be in the same cross-section as the middle rivets in the webs, thus increasing the available section to 14.74 square inches.

The insufficient provision frequently made for counter-stresses in railroad bridges is discussed in a paper by H. S. PRICHARD, in Transactions American Society of Civil Engineers, vol. 42, page 547, Dec., 1899.

ART. 90. LOWER CHORD SECTIONS.

SPECIFICATION. — For single-track spans the two panels of the lower chord, at each end, shall preferably be made rigid members.

If the wind stresses be neglected, the required net area for the lower chord member *cd* is 641 700/15 000 = 42.78 square inches, while if they be included, the net area is 824 900/19 000 = 43.42 square inches (Art. 83). The larger area must be taken. Four eye-bars $8'' \times 1\frac{3}{8}''$ give 44 square inches, and are therefore chosen.

In the case of *de* the greater sectional area is obtained by omitting the wind stresses, its value being 769 900/15 000 = 51.33 square inches. Four eye-bars, $8'' \times 1\frac{5}{8}''$, are needed whose combined sectional area is 52 square inches.

The stresses which govern the design of *ab* and *bc* are the same, and hence a single member may be extended from *a* to *c*. The required net area is 385 000/15 000 = 25.67 square inches. Let *ac* be composed of two built channels, like Fig. 90. Since the eye-bar heads of the 8-inch eye-bars are 17 inches deep according to the handbook, let the web plates be made 18 inches deep so as to avoid cutting the angles in order to pass the eye-bar heads at *c*. Selecting 2 web plates $18'' \times \frac{1}{2}''$, and 4 angles $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{1}{2}''$, the rivets in the end pin plates can be so arranged as not to deduct more than 3 rivet holes in each web plate and one in each angle, giving a net area of 26 square inches.

The specifications also require that, if the unit stress due to the weight of a member is greater than 10 percent of the safe value allowed, the sectional area must be increased. According to the first method given in Mechanics of Materials, Art. 118, the stress in an eye-bar 8 inches deep and 25 feet long, due to its own weight, is found to be 1187 pounds per square inch, which is less than the above limit.

The formula is $$S_1 = \frac{Mc}{I + \frac{nPl^2}{mE}},$$

in which M is the bending moment of the flexural forces, c the distance from the neutral surface to the outer fiber in which the tension S_1 occurs, I the moment of inertia of the cross-section, n and m numbers depending upon the arrangement of the ends and the kind of loading, P the longitudinal tensile force, l the length of the member, and E the coefficient of elasticity of the material. The flexure of the eye-bar under its own weight corresponds to that of a simple beam uniformly loaded; and hence $m/n = 9.6$. The same result is obtained for any thickness of bar, and hence a single determination only is required for all bars of the same depth and length. The second and more accurate method given in the same article gives a stress of 1176 pounds per square inch. The value of E for soft steel is taken as 26 000 000 pounds per square inch.

The results of some experiments on the relative strength of eye-bars and built members of various forms of section are given in a paper entitled 'Recent Tests of Bridge Members,' by J. E. GRENIER, in Transactions American Society of Civil Engineers, vol. 38, page 41, Dec., 1897.

ART. 91. DIAMETERS OF PINS.

SPECIFICATION. — The stress in the outer fibers of pins shall not exceed 25 000 pounds per square inch, the points of application of the stresses in the connecting members being taken at the centers of bearings. In designing all pin-connected work ample clearance for packing must be provided, and ample room must be left for assembling members in confined spaces. Lower chords are to be packed as closely as possible, and in such a manner as to produce the least bending moments on the pins, but adjacent eye-bars in the same panel must never have less than a one-half-inch space between them, in order to facilitate painting. The various members attached to any pin must be packed as closely as practicable, and all interior vacant spaces must be filled with steel fillers, where their omission would permit of the motion of any

member on the pin. All bars are to lie in planes as nearly as possible parallel to the central plane of the truss, no divergence exceeding one-eighth of an inch to the foot being permitted.

In order to find the bending moment in the pin at *d* it is necessary to determine provisionally the thickness of the pin plates for the post *Dd* and the diagonals *Cd* and *Ed*. As the specifications do not permit countersunk rivet heads in metal less than $\frac{7}{16}''$ thick, one pin plate of this thickness will be placed on each side of the web of each channel in the post. These plates and the webs will furnish sufficient bearing area for a pin whose diameter is not less than $5\frac{1}{2}$ inches. The pin cannot be less than $8 \times 15\,000/22\,000 = 5.46$ inches for 8-inch eye-bars, in order to provide adequate bearing area for the eye-bars. Assuming that the pin will not exceed 6 inches in diameter, it is estimated that a $\frac{9}{16}''$ pin plate will be required on each side of the webs of the channels composing *Cd*, in order that the net sectional area at the pin hole shall not be less than 40 percent in excess of that of the net section elsewhere. The diagonal *Ed* is connected to the pin by means of pin plates only whose combined thickness on each side is 1 inch, the angles being cut off so as not to interfere with the post.

The horizontal forces in the chord bars and diagonals produce flexure in the pin in a horizontal plane, the bending moment being designated by M_h, while the vertical forces in the diagonal and post produce a moment M_v, their resultant being

$$M = \sqrt{M_h^2 + M_v^2}.$$

In order to reduce M_v, the bearings of the diagonal are placed outside of and next to those of the post. The eye-bars in the chord alternate in direction so as to allow space between adjacent bars of the same panel for painting. To reduce M_h, one of the smaller bars is put on the outside. The arrangement, or packing, of the eye-bars, etc., on one side is indicated in Fig. 119.

A clearance of $\frac{1}{16}''$ is allowed between eye-bars, and $\frac{1}{8}''$ where one or both of the adjacent members contain countersunk rivet heads. The stresses marked on the chord bars are the safe values for their actual sections. The stress in *Cd* is that required to make the algebraic sum of all the horizontal forces acting on the pin equal to zero. The straight sides of the equilibrium polygon are drawn by considering the stresses applied at the centers of the respective bearings, as specified,

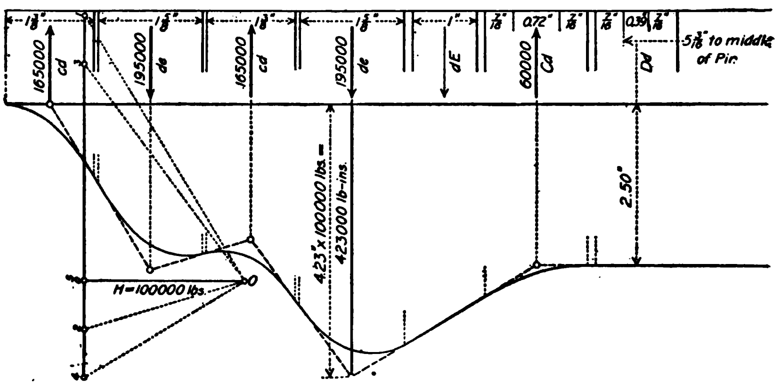


Fig. 119.

while the curved lines give the form of the diagram when the stresses are regarded as uniformly distributed on the bearings. The curves are parabolas, the points of tangency lying in the ordinates through the sides of the eye-bars, and other members (Part II, Art. 10). The original diagram was drawn full size for the linear dimensions, and the pole distance made equal to 100 000 pounds. The ordinate at the post bearing, where M_v is a maximum, measured 2.50 inches, making

$$M_h = 2.50 \times 100\,000 = 250\,000 \text{ pound-inches,}$$

$$M_v = 744\,000 \times 1.49 = 110\,900 \text{ pound-inches,}$$

74 400 pounds being the vertical component of the stress in *Cd* which corresponds to the horizontal component of 60 000

pounds, while 1.49 inches is the distance between the centers of bearing of the diagonal and post. Both horizontal and vertical moments are uniform between the two sides of the post, as there are no forces acting on the pin within those limits. The resultant M is 273 400 pound-inches, but since this value is less than M_h at the inner bar of *dc* where there is no M_v, the maximum moment is 423 000 pound-inches. According to the handbook this requires a pin $5\frac{5}{8}$ inches in diameter, whose resisting moment is 436 800 pound-inches.

In a similar manner the bending moments are found in the pin at *c*, it being estimated that two pin plates are required for each half of the stiff chord member *bc*, one $\frac{5}{8}''$ and the other $\frac{1}{2}''$ in thickness. The greatest value of M_h is 446 000 pound-inches at the post bearing, while M_v is 295 000 pound-inches, making $M = 529\ 500$ pound-inches. A 6-inch pin whose resisting moment is 530 200 pound-inches will therefore be required.

If the outer eye-bars in *cd* be reduced in thickness to $1\frac{1}{8}$ inches and the inner ones increased to $1\frac{5}{8}$ inches, the other members remaining the same, M_h is reduced to 340 000 pound-inches, and M to 443 500 pound-inches, thus requiring a pin $5\frac{3}{4}$ inches in diameter. This change makes a still larger reduction in the moments on the pin at *d*, and indicates the method by which the diameter of the pin may be brought within given limits when desired.

The bending moments on the pins at *a* and *B* need not usually be found, as they are smaller than those at *c* and *d*. The pins at those panel points will, however, be made the same size, so as to reduce the total thickness of pin-plates otherwise required. Pins 6 inches in diameter will then be adopted at *a*, *c*, *d*, and *B*. Those at *C* and *D* will be designed after the upper chord sections are determined.

ART. 92. UPPER CHORD SECTIONS.

SPECIFICATION. — In members subject to compression, rivets shall be so spaced that they shall not be farther apart in the direction of the stress than sixteen times the thickness of the thinnest external plate connected, and not more than fifty times that thickness at right angles to the direction of the stress.

The standard diameter of the head of an 8-inch eye-bar when the pin does not exceed 6 inches is 17 inches, according to one of the handbooks. If the eye-bars of Bc are placed inside of the upper chord, the web plates must be at least 18 inches deep to provide ample clearance. The width of the chord must also be determined at this panel point. For a single-track bridge of 175 feet span a composition like that shown in Fig. 96 is appropriate. In Art. 88 the distance back to back of the channels in the posts was found to be $10\frac{3}{8}$ inches, and hence those in the suspender will be spaced the same distance. The pin plates on the outside of the suspenders are estimated to be $\frac{11}{16}''$ thick. Considering one side only for simplicity, and allowing $\frac{1}{8}''$ for a clearance next to the pin plate, $1\frac{7}{8}''$ for the thickness of one eye-bar in Bc, $\frac{1}{4}''$ for another clearance, $\frac{3}{8}''$ for a jaw plate, and $\frac{1}{2}''$ for a pin plate, both connected to the inside of the upper chord web, and $\frac{1}{2}''$ for the thickness of web plate, the distance from the center of the suspender to the outside of the chord web plate, or to the backs of the angles, is found to be $9\frac{1}{2}$ inches, and if the angles be $3\frac{1}{2}$ inches wide, the cover plate must be 26 inches wide. The lines of rivets connecting it to the angles are 23 inches apart, and hence the cover plate must be $\frac{7}{16}''$ thick to satisfy the requirement of the specifications regarding the maximum spacing of rivets at right angles to the direction of the stress in compression members.

Since the specified unit stress involves the radius of gyration, an approximate value must be assumed. A convenient rule

makes the radius of gyration about a horizontal axis equal to four-tenths of the depth out to out. This depth is estimated to be 19.44 inches, making $r = 7.78$ inches, $l/r = 38.6$, $p = 13\,510$ pounds per square inch, and the required sectional area

$$641\,700/13\,510 = 47.50 \text{ square inches.}$$

The composition of the section is as follows:

1 cover plate, $26'' \times \frac{7}{16}''$	11.37 square inches.
4 angles, $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{7}{16}''$	11.48
2 web plates, $18'' \times \frac{7}{16}''$	15.76
2 flats, $5'' \times 1''$	10.00
Total	48.61

Placing the backs of the angles $\frac{1}{8}''$ beyond the edges of the web plates, the center of gravity is found to be $0.21''$ above the center of the web plates. The pin may accordingly be placed with its axis passing through the centers of the web plates, for it is found that an eccentricity of $0.24''$ would just make the negative moment due to the maximum direct compression equal in magnitude to the positive bending moment produced by the weight of the chord.

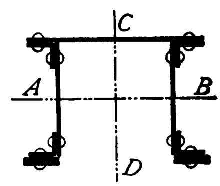


Fig. 120.

The following computation gives the moment of inertia with reference to the neutral axis AB in Fig. 120:

1 cover plate, $\frac{1}{12} \times 26 (\frac{7}{16})^3$	=	0.2
$11.37 (0.22 + 0.125 + 9.0)^2$.	=	992.9
4 angles, 4×3.26	=	13.0
$11.48 (9.125 - 1.04)^2$	=	750.4
2 web plates, $2 \times \frac{1}{12} \times \frac{7}{16} \times 18^3$	=	425.2
2 flats, $2 \times \frac{1}{12} \times 5 \times 1^3$	=	0.8
$10 (9.125 + 0.5)^2$	=	926.4
		3108.9
$48.61 (0.21)^2$	=	2.1
I	=	3106.8 inches^4.

For convenience the moment of inertia is first computed for a horizontal axis through the centers of the web plates and then reduced to the neutral axis. The radius of gyration is

$$r = \sqrt{\frac{3106.8}{48.61}} = 8.00 \text{ inches,}$$

and the revised area is found to be 47.25 square inches.

As the stress in *CD* is larger than that in *BC*, and its additional sectional area will be placed in the sides, leaving the cover plate and flats unchanged, its radius of gyration will be a little smaller than that for *BC*, and hence $r = 7.78$ inches will again be assumed. The approximate sectional area for *CD* is then $769\,900/13\,510 = 56.99$ square inches. Using the same composition as for *BC* except to increase the thickness of the web plates to $\frac{11}{16}''$, the area is 57.61 square inches. The moment of inertia is then computed to be 3350.2 inches^4, the radius of gyration 7.71 inches, and the revised required area 57.07 square inches. The use of $\frac{9}{16}''$ angles and $\frac{5}{8}''$ web plates would also satisfy the requirements, but the gross area is then 58.37 square inches. The former composition also has the advantage in not requiring the pin plates at *C* and *D* to connect to the angles, but only to the web plates because the entire increment of chord stress is taken by the web plates. The section for *DE* equals that for *CD*.

Inspection shows that the moments of inertia around the neutral axis *CD* in Fig. 120 are respectively greater than those computed for the sections of both chord members, and hence the values of r determined above are the least radii of gyration required in the column formula.

The diameter of the pin at *C* may now be determined. The pin plates on the post *Cc* are $\frac{7}{16}''$ thick, and those on the diagonal *Cd* are approximately $\frac{9}{16}''$ thick, one plate being placed on each side of each channel web. One $\frac{3}{8}''$ pin plate is also needed on

the outside of each web plate of the chord if the pin is not less than 5 inches in diameter. Remembering that the channels in the post are spaced $10\frac{3}{8}''$ back to back, and allowing for a clearance of $\frac{1}{8}''$ between the adjacent pin plates of the posts and diagonals, the distance from the center of one of the chord bearings to that of the diagonal is 2.672 inches, and from the latter to the center of the adjacent post bearing is 1.747 inches. Since the chord section is continuous past the pin, the maximum bending moment on the pin occurs when the stress in the diagonal is a maximum. The full strength of the net section of Cd is $21.16 \times 15\,000 = 317\,400$ pounds. Its horizontal component of 199 200 pounds equals the corresponding increment of chord stress, and its vertical component of 241 400 pounds is the corresponding stress in the post. $M_h = 99\,600 \times 2.672 = 266\,100$ pound-inches, and $M_v = 120\,700 \times 1.747 = 210\,900$ pound-inches, their resultant M being 339 500 pound-inches. This requires a pin whose diameter is $5\frac{1}{4}$ inches. The same size will be adopted for that at the panel point D.

In case it be desired to reduce the thickness of the flats, an alternative section may be designed by increasing the horizontal legs of the angles to a width of 5 inches. Since the gage of the 5-inch leg for a single row of rivets is 3 inches, a flat 6 inches wide will not extend beyond the back of the angle. This width requires a thickness of $\frac{3}{4}$ inch for the flats and increases the total chord section by 0.32 square inch, but reduces the eccentricity of the neutral axis to 0.18 inch. If, however, the flats be taken 7 inches wide, their required thickness is $\frac{5}{8}$ inch, the corresponding increase in chord section being only 0.08 square inch, while the eccentricity is increased to 0.24 inch.

ART. 93. SECTION OF INCLINED END POST.

SPECIFICATION. — The inclined end post must be so proportioned that the algebraic sum of the stresses per square inch resulting from the direct com-

pression and the maximum bending moment due to the wind pressure shall not exceed 19,000 pounds per square inch. Every column that acts as a beam also must have solid webs at right angles to each other, as no reliance shall be placed on lacing to carry a transverse load down the column.

The maximum direct compression in the end post aB is not quite as large as that in BC, but its length is greater, being 477.9 inches. Its sectional area will therefore not differ much from that of BC. Using the value obtained for BC of $r = 8.00$ inches, $l/r = 60.0$, $p = 11\,840$ pounds per square inch, and the approximate sectional area is $613\,000/11\,840 = 51.78$ square inches. The following composition is adopted, as a test shows that no revision is needed:

1 cover plate, $26'' \times \frac{7}{16}''$	11.37 square inches.
4 angles, $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{7}{16}''$	11.48
2 web plates, $18'' \times \frac{9}{16}''$	20.24
2 flats, $5'' \times 1''$	10.00
Total	53.09

The wind stresses, according to the specifications, are not large enough to affect the area required to resist flexure in the plane of the truss, but must be considered in computing the stresses due to transverse flexure.

The end posts form a part of the portal which resists the wind pressure carried by the upper lateral system to the portal strut. The end posts bend in the plane containing their center lines. It is necessary then to compute the unit stress in the outer fiber of each end post due to its combined action as a column under the given direct compression and as a beam subject to the bending moment produced by the wind loads, and to compare it with the greatest allowable unit stress. If it exceeds the allowable value, the sectional area must be increased until the unit stress falls within the given limit.

The moment of inertia with reference to the axis CD in Fig. 120 is computed as follows, it being remembered that the backs of the angles are 19 inches apart:

1 cover plate, $\frac{1}{12} \times \frac{7}{16}(26)^3$	. . . .	$= 640.8$
4 angles, 4×3.26	. . . .	$= 13.0$
$11.48(9.5 + 1.04)^2$	. . . .	$= 1275.3$
2 web plates, $2 \times \frac{1}{12} \times 18(\frac{9}{16})^3$	. . . .	$= 0.5$
$20.24(9.5 - 0.28)^2$	. . . .	$= 1720.6$
2 flats, $2 \times \frac{1}{12} \times 1 \times 5^3$	. . . .	$= 20.8$
$10.00(9.5 + 2.0)^2$	. . . .	$= 1322.5$
I'	. . . . .	$= 4993.5$

This computation indicates that the center line of each flat and the rivet line of the corresponding angle lie in the same vertical plane. Any eccentricity in the connection of the flats would develop secondary bending stresses. The radius of gyration with reference to the same axis is

$$r = \sqrt{\frac{4993.5}{53.09}} = 9.70 \text{ inches.}$$

Let the upper part of the end post be regarded as fixed at the lower portal strut by the portal bracing, and the lower end as fixed at the pin by the pedestal and the end floor beam. The drawing of the floor beam gives $4' \ 8\frac{3}{8}''$ as the distance from the base of rail to the bottom of the floor beam, and $10\frac{1}{2}''$ from the bottom of floor beam up to the pin center. The clear head room required is 23 feet, and hence the distance from the center of the lower chord to the top of the clearance must not be less than $26' \ 4\frac{7}{8}''$. Allowing from 10 to 12 inches for the width of the lower portal strut, the inclined distance from the lower pin of an end post to the

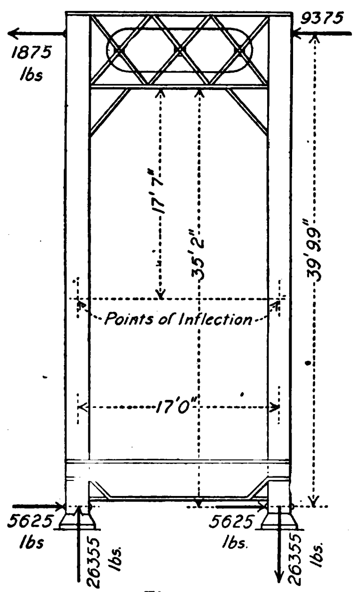


Fig. 121.

bottom of the portal strut is readily found by means of a diagram to be just 35 feet. Let the distance be taken as 422 inches. The point of inflection is at the middle of this length.

Fig. 121 indicates the action of forces on the portal, it being assumed that the reactions at the feet of the end posts are equal. The transverse bending moment in each end post is therefore $5625 \times 211 = 1\,186\,900$ pound-inches. By the first method given in Mechanics of Materials, Art. 117, the unit stress in the outer fiber is found to be

$$S_1 = \frac{Mc}{I - \frac{Pl^2}{6E}} = \frac{1\,186\,900 \times 14.0}{4993.5 - \frac{666\,700 \times \overline{422}^2}{6 \times 26\,000\,000}} = 3920 \text{ lbs. per sq. in.,}$$

making the maximum compressive stress equal to $666\,700/53.09 + 3920 = 16\,480$ pounds per square inch. As this value does not exceed the specified limit of 19 000 pounds per square inch, no change is required in the composition of the end post.

The general formula given in the text-book just mentioned is the same as that quoted in Art. 90, except that the sign in the denominator is changed to minus. The form in which it is here given is adapted to the special case of the end post whose elastic line corresponds to that of one-half of a beam whose ends are fixed and loaded at the middle. Accordingly, $m = 192$, $n = 8$, and the l in the general formula equals $2\,l$ in the formula given in this paragraph.

A beam whose span is l, fixed at both ends, supporting a concentrated load Q at the middle, and subject to a longitudinal compression P, has a maximum compressive stress at the ends, or at the load, whose value is

$$S = \frac{P}{A} + \frac{Qc}{2\,\beta I}\tan\tfrac{1}{4}\beta l,$$

in which A is the area of cross-section of the beam, c the

distance from the neutral surface to the outer fiber whose unit stress is S, and $\beta = (P/EI)^{\frac{1}{2}}$, in which E is the coefficient of elasticity of the material. Applying this formula to the case of the end post, and remembering that l in this formula equals twice that used in the special approximate formula of the preceding paragraph, the true maximum compressive stress is found to be

$$S = \frac{666\,700}{53.09} + 3654 = 16\,210 \text{ pounds per square inch,}$$

or 270 pounds less than the approximate value first obtained.

ART. 94. LATERAL BRACING.

SPECIFICATION. — All lateral bracing shall be made of shapes which can resist compression as well as tension. In detailing struts composed of four angles with a single line of lacing, the clear distance between backs of angles shall never be made less than three-quarters of an inch, in order to permit the insertion of a small paint-brush. The stiff diagonals of the lower lateral system, of which there shall be two in each panel, shall be riveted rigidly to the stringers where they cross them, so as to transfer in an effective manner the thrust of braked trains to the truss posts without causing the floor beams to bend horizontally. In designing short members with riveted connections the sectional area of the piece shall be increased from 10 percent for 6″ × 3½″ angles to 25 percent for equal-legged angles beyond the theoretical requirements for the direct stresses, so as to compensate for the secondary stresses due to the eccentric grip of the rivets.

The most approved type of laterals for the upper system of through spans consists of two pairs of angles with one system of lacing between them, the depth of the member being equal to that of the upper chord. As the computed wind stress is only 13 300 pounds in one of the end laterals under the assumption that it resists the entire shear in the panel, it is clear that the stress alone cannot determine the section to be used. Since the principal duty of the lateral bracing is to resist the lateral vibration caused by the live load, and to hold the chord in line, it is important that they should have ample section to insure the

necessary stiffness. By using $\frac{3}{4}''$ rivets in the connections, and $\frac{5}{8}''$ rivets in the lacing, the angles may be reduced in size to $3'' \times 2\frac{1}{2}'' \times \frac{3}{8}''$, the longer legs being horizontal. For the required depth the lattice bars must be at least $\frac{3}{8}''$ thick, and hence the backs of the angles are spaced $\frac{3}{4}''$ apart. This makes the least radius of gyration 1.47 inches, and $l/r = 108.2$, the length from the center, where the laterals are riveted to a common connecting plate to the end connection, being about 159 inches. If the entire stress be resisted by one lateral as a column, the required area is 1.66 square inches, while only 0.89 square inch net section is needed for the same amount of tension. The section may then be adopted, although there is considerable excess of strength. If only two angles were laced together, they would have to be $4\frac{1}{2}'' \times 3''$ in size in order that the ratio l/r should not fall below 120, as specified for compression members. When the ratio falls below this limit, or when only single angles are used, they can be regarded merely as tension members, and the lateral system becomes correspondingly less efficient.

The net strength of the lateral when one rivet hole is deducted in each leg of the angles is $5.24 \times 15\,000 = 78\,600$ pounds, and hence the connections require 16 shop rivets or 20 field rivets $\frac{3}{4}''$ in diameter.

The stresses in the upper lateral struts are so small that their section is not determined theoretically. The student should consult some standard plans for bridges governed by approximately similar conditions and adopt the composition shown. See Plate IV for an example of a lateral strut which also forms the upper chord of the sway bracing.

Since the lower laterals are to be riveted to the lower flanges of the stringers, it is estimated that the greatest distance between the centers of connections is about 94 inches. In the first panel

the stress corresponding to the total shear is 80 000 pounds. Let the laterals be so designed as to resist either this entire amount in tension or half of this amount in both tension and compression, under the specification for alternate stresses, the wind stresses being treated as live-load stresses, but without any impact allowance. Placing two angles $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{5}{8}''$ back to back, the required area for compression is 4.29, while that for tension is 2.67 square inches, making a total of 6.96 square inches. The gross area furnished is 7.96 square inches, which covers an allowance of more than 12 percent for the effect of eccentric connections. Deducting one rivet from each leg of the angles for $\frac{7}{8}''$ rivets, the net section is 5.46 square inches, while a tension of 80 000 pounds requires only 5.34 square inches.

In a similar manner the laterals in the second panel require two angles $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{7}{16}''$, while in the third and fourth panels two angles $3\frac{1}{2}'' \times 3'' \times \frac{3}{8}''$ may be used.

The connections require 13 shop or 16 field rivets in the first panel, 9 shop or 12 field rivets in the second panel, and 8 shop or 10 field rivets in the third or fourth panels, in order to develop the full strength of the net sections of the laterals.

The results of tests made by J. E. GREINER on the relative strength of single angles when connected by one or by both legs are recorded in Transactions American Society of Civil Engineers, vol. 38, page 63, Dec., 1897, while those made by C. F. LOWETH are given in Journal of the Association of Engineering Societies, vol. 8, page 268, May, 1889.

In order to comply with the specification which aims to prevent horizontal bending in the floor beams, due to the thrust of braked trains, WADDELL further specifies that the lateral diagonals and the stringers are to be made to form complete horizontal trusses by running angles between the stringers at the level

of the bottom flanges. Such angles are represented diagrammatically by the lines mm' and nn' in Fig. 122. As the equivalent uniform live load for a panel length of 25 feet is 9850 pounds per linear foot per track, the total thrust of the braked train per panel is $9850 \times 25 \times 0.20 = 49\,250$ pounds, the coefficient of friction being 20 percent (Art. 83). Dividing this between the two stringers, and assuming that the entire stress is taken by the truss $dnn'd'$, the stress in nn' is 16 800 pounds, and that in nd or in $n'd'$ is 26 800 pounds. If the truss $cmm'c'$ is assumed to act in conjunction with the other, the stresses will be divided accordingly. The smallest laterals have sufficient strength to take the stress of 26 800 pounds, either in tension or compression. A single angle, $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$, having a gross sectional area of 2.48 square inches, and a net area of 2.10 square inches, is found to have an excess of strength to provide for the eccentricity of its connections, even if it be subject to the entire stress of 16 800 pounds. The stress of 26 800 pounds requires 5 shop rivets or 6 field rivets, while that of 16 800 pounds requires 3 shop rivets or 4 field rivets. A detail of this kind and the connection of a lateral to one of the stringers may be seen on Plate VI.

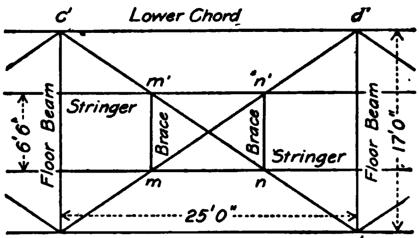


Fig. 122.

Art. 95. Portal and Sway Bracing.

SPECIFICATION. — All through spans shall have stiff portal bracing at each end, connected rigidly to the end posts. The bracing shall be made as deep as the specified clear head room will allow. When the height of the trusses is great enough to permit it, there shall be used at each panel point a rigid bracing frame riveted to the top lateral strut, and to the posts, and carried down to the clearance line. When the truss depth is not great enough for

this detail, corner brackets of proper size, strength, and rigidity are to be riveted between the posts and the upper lateral struts.

Fig. 123 gives a sketch of the general arrangement of the portal bracing to be designed, and the principal dimensions required to compute the bending moments to be resisted by the flanges of the lattice girder. The upper flange of the bracing

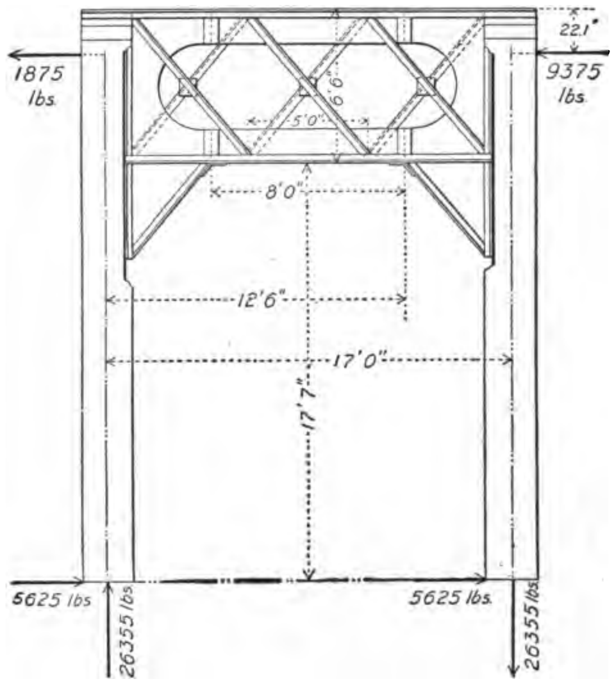


Fig. 123.

extends beyond the upper chords of the truss. The given forces are those obtained in designing the end post, and the reactions at the bottom of the figure are applied at the points of inflection of the end posts.

Let the shear be divided equally between the two diagonals cut by any section parallel to the end posts, and let the diagonals

be designed according to the specification for alternate stresses, the wind stresses being treated as live-load stresses for the same reasons as those given in the preceding article. The stress in each angle is then $13\,180 \times 1.2875 = \pm 16\,970$ pounds. Assuming one angle $3\frac{1}{2}'' \times 3'' \times \frac{7}{16}''$, and the distance between centers of connections as about 38 inches, the area for compression is found to be 1.45 square inches, while that for tension is 1.13 inches, making the total a little less than that of the angle, which is 2.65 square inches. The number of $\frac{7}{8}''$ rivets required in the connection to the flange is $2 \times 16\,970/6610 = 6$.

This result indicates that the plate in the flange must be 17 inches wide. Assuming two flange angles $5'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$, and neglecting the plate on account of the splice at the section where the flange stress is a maximum, the effective depth is 76.1 inches. With the forces shown in Fig. 123, the bending moment in a section at the end of the left bracket is $-157\,500$ pound-inches, at the middle it is $+1\,107\,500$ pound-inches, and at the right bracket it reaches its maximum of $+2\,372\,500$ pound-inches, the center of moments being in the neutral axis of the upper flange angles. This requires a net section of 2.03 square inches in the lower flange. When the forces are reversed in direction, the upper forces exchanging sides, the bending moment in the same section is $-2\,531\,500$ pound-inches, which requires a flange area of 2.22 square inches. The sum of the two areas is 4.25 square inches.

In a similar manner the areas required for the tension and compression in the upper flange are 2.33 and 2.87 square inches, or a total of 5.20 square inches. The gross area of the assumed angles is 6.10, while the net area is 4.60 square inches when a rivet hole is deducted from both legs of the angles, and 5.35 square inches when deducted from only one leg in each angle. The smaller net area applies to the lower flange and the larger one to the upper flange, the difference being due to

the connection of the lower flange angles to the bracket angles. The angles, therefore, have ample section to cover these requirements. The ratio l/r equals 75.3 for the upper flange, which is not stayed between the trusses in a direction perpendicular to the plane of the portal. By increasing the longer legs of the angles to 6 inches, this ratio may be reduced to 61.9, thus considerably adding to the lateral stiffness of the flange.

The statement made in the preceding article with reference to the design of the lateral struts applies to the entire intermediate sway bracing, of which the strut forms a part. The student should consult standard plans and make a comparative study of the details of the sway bracing. (See Plates IV and VII, and Art. 82.)

ART. 96. PIN PLATES.

SPECIFICATION. — Rivets shall not be countersunk in plates less than seven-sixteenths of an inch in thickness.

Pin plates shall be used at all pin holes in built members for the double purpose of reinforcing for the metal cut away and reducing the unit pressure on pin and bearing to or below the specified limit. They shall be of such size as to distribute properly, through the rivets, the pressure carried by such plates to both flanges and web of each segment of the member; and they shall extend at least six inches within the tie plates of said member, so as to provide for not less than two transverse rows of rivets there.

It is always better, whenever practicable, to avoid cutting away the ends of channels, but if they must be trimmed, the ends must be reënforced so that the strength of the member shall not be reduced by the trimming.

In riveted tension members, the net section through any pin hole shall have an area 40 percent in excess of the net sectional area of the body of the member. The net section outside of the pin hole along the center line of stress shall be at least 70 percent of the net section through the pin hole.

In designing the pin plates of the various members of the truss, it is necessary to observe not only the specification printed at the head of this article, but also the general one in Art. 83, which requires that in all main members having an excess of section above that called for by the greatest combination of

stresses, the entire detailing is to be done for the utmost working capacity of the member, and not merely for the greatest total stress to which it may be subjected.

The maximum pin bearing at the bottom of the post *Cc* equals the maximum vertical shear in the diagonal *Bc*, and according to the rule just quoted, the value to be used in designing the pin plates of the post is the vertical component of the full working strength of *Bc*, which is $30 \times 15\,000/1.2847 = 350\,300$

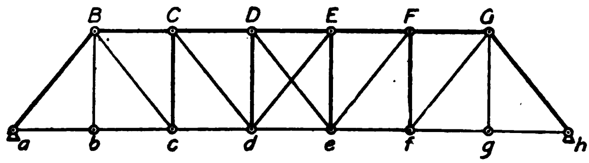


Fig. 124.

pounds, the sectional area of *Bc* being 30 square inches, and 1.2847 the secant of the angle which it makes with the vertical. As the diameter of the pin is 6 inches (Art. 91), the bearing area required on each side of the post is $350\,300/(2 \times 6 \times 22\,000) = 1.327$ inches. The thickness of the channel web is 0.524 inch, and hence two pin plates are required whose thicknesses are respectively $\frac{7}{16}$ and $\frac{3}{8}$ of an inch. The outer pin plate cannot be less than $\frac{7}{16}$ inch, according to the specifications, since its rivets must be countersunk. If both plates be extended the same distance above the pin, the number of rivets required to connect them will be determined entirely by their bearing value in the channel web, or $0.875 \times 0.524 \times 22\,000 = 10\,090$ pounds per square inch. The distribution of stresses between the pin plates and channel is in direct proportion to their respective bearings on the pin, and hence the stress taken by both pin plates is $0.813 \times 175\,100/1.337 = 106\,500$ pounds. Their full bearing value, however, is $0.813 \times 6 \times 22\,000 = 107\,300$ pounds, and therefore this stress is to be used according to the specifications. The number of rivets required is then

107 300/10 090 = 11. Fig. 125 shows the arrangement of the rivets. The outer pin plate on each side is extended to the foot of the post so as to act as a washer between the channel and the eye-bar *Bc*, and additional rivets are placed below the pin to keep the parts in contact.

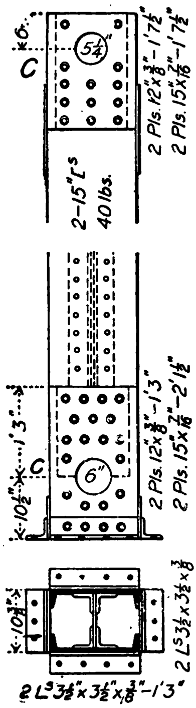


Fig. 125.

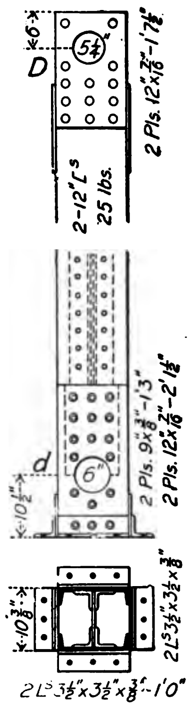


Fig. 126.

At the upper panel point the maximum bearing value on the pin is the full working strength of the post *Cc*, which is 23.52 × 11 110 = 261 300 pounds. This stress requires a bearing on each side of the post of 1.131 inches. Since the rivets in the outer pin plate must be countersunk, its thickness cannot be less than $\frac{7}{16}$ inch, and if this thickness be adopted, the inner plate must be $\frac{3}{8}$ inch thick, the minimum allowed. The full bearing value of both plates is 0.813 × 5¼ × 22 000 = 93 900 pounds, requiring 10 rivets to transmit their stress into the channel web. A symmetrical arrangement requires 11 rivets, as indicated in Fig. 125. The sizes of the pin plates and their riveting for the post *Dd* are given in Fig. 126.

Since the suspender *Bb* is a tension member, its net sectional area at the pin hole must be 40 percent in excess of the net area in its main body. The area for each side is therefore 13.71 × 1.40/2 = 9.60 square inches. The simplest arrangement is to use one pin plate 14″ × $\frac{9}{16}$″, giving a net area at the

pin of 10.24 square inches, but on computing the distance required beyond the pin hole it is found to be $6\frac{7}{8}$ inches, which exceeds the limit allowed by the upper chord. Using one pin plate $12'' \times \frac{7}{16}''$ on the outside, and another $9\frac{1}{2}'' \times \frac{3}{8}''$ on the inside, the net area obtained is 9.67 square inches. The net areas of the pin plates are 2.62 and 1.31 square inches, while their full tensile strengths are 39 300 and 19 650 pounds respectively. It is found that their bearing on the pin is below the specified limit. The value of a rivet in single shear is 6610 pounds, and its bearing in the web of the channel is $0.875 \times 0.513 \times 22\,000 = 9870$ pounds. If the inner plate be shorter than the outer one, it requires $19\,650/6610 = 3$ rivets. Their bearing value in the web is $9870 \times 3 = 29\,610$ pounds, leaving a balance of $19\,650 + 39\,300 - 29\,610 = 29\,340$ pounds to be taken by the additional rivets in the longer plate, and this requires $29\,340/6610 = 5$ rivets. If both plates have the same length, the number of rivets needed is $58\,950/9870 = 5$ rivets. To reduce the effect of eccentricity, both plates are lengthened somewhat beyond the limits indicated by the preceding computations. (See Fig. 127.) It must be remembered that in designing this member an allowance was made for two rivet holes in the flange and two in the web of each channel, and hence only two rivets are placed in any section below the top of the tie plates which are shown on the sides of the member. The distance beyond the pin is $9.67 \times 0.70/1.326 = 5\frac{1}{8}$ inches, according to the specifications.

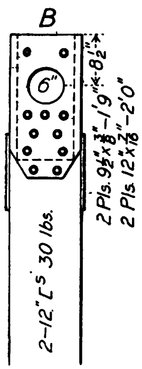


Fig. 127.

The net area at the pin holes in the diagonal *Cd* must not be less than $21.16 \times 1.40 = 29.62$ square inches, or 14.81 square inches for each side of the member. By turning outward the flanges of the channels the cutting needed to avoid interference

with other members of the truss is reduced to a minimum. At the lower end the flanges must be cut off entirely next to the eye-bars, and hence two pin plates, each $\frac{9}{16}$ inch thick, are required on each side. The net areas of the channel and of the inner and outer pin plates are 6.48, 5.06, and 3.38 square inches respectively, making a total of 15.86 square inches. Since the bearing of the rivets in the web of the channel, whose thickness is 0.72 inch, is greater than the double shear, the number of rivets in each pin plate is governed by single shear. The inner plate requires $5.06 \times 15\,000/6610 = 12$ rivets, and the outer one $3.38 \times 15\,000/6610 = 8$ rivets. But the full bearing value of the inner pin plate is only 74 250 pounds, which being less than its full tensile strength will slightly modify the

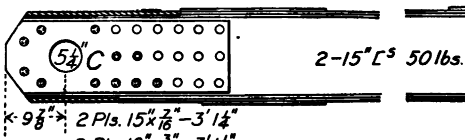


Fig. 128.

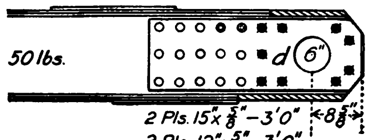


Fig. 129.

distribution of stress beyond the pin. As several rivets are placed there to keep the plates in contact, they will have ample strength to transfer the stress. Since this member is also subject to compression, both pin plates will be made the same length in order to give additional stiffness to its forked ends. On account of this extension in length the number of rivet lines is reduced from four to three, and this requires a net area at the pin of 15.82 square inches, and a change in the thickness of the pin plates to $\frac{5}{8}$ inch in order to conform to the specification quoted from Art. 83. The required number of rivets is increased by one for each plate. The distance beyond the pin is $15.86 \times 0.70/1.97 = 5\frac{5}{8}$ inches. (See Fig. 129.)

At the upper end of *Cd* the flanges of the channels need only to be cut down to $2\frac{1}{4}$ inches, thus leaving a clearance of $\frac{1}{4}$ inch

between them and the upper chord with plates. The required sizes of the pin plates are marked on Fig. 128. The net areas of the channel, and of the inner and outer pin plates, are respectively 9.37, 4.27, and 2.53 square inches, making a total of 16.17 square inches. The stresses taken by the pin plates are 64 100 and 38 000 pounds, while the required numbers of rivets are 10 and 6 respectively. The plates are extended farther so as to pass the tie plates.

As shown in Fig. 130, the angles in the diagonal *dE* have to be cut off entirely one foot from the pin center in order to avoid interference with the post channels of *Dd*. The entire stress must, therefore, be carried to the pin by the pin plates. The full strength of *dE* is $17.88 \times 9210 = 164\,700$ pounds, since the area was determined by the compressive stress. The linear bearing on the pin for each side of the member cannot be less than $164\,700/(2 \times 6 \times 22\,000) = \frac{5}{8}$ inch. The full compressive strength of the member does not determine the net area required at the pin hole. If it be assumed that the entire shear in the panel causes tension in only one of the diagonals, the net area would be $152\,500/15\,000 = 10.17$ square inches.

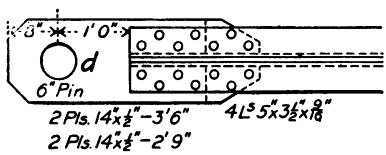


Fig. 130.

The corresponding area at the pin is, therefore, 7.12 square inches for each side, and if the pin plates be taken 14 inches wide, the total thickness must be 0.89, or say 1 inch, to allow something for excess of section in the member. Let two plates be used, each one-half an inch thick. The shorter one requires $41\,200/6610 = 7$ rivets, and both of them need $82\,400/6610 = 13$ rivets. The pin plates are to extend $7.12 \times 0.70/1 = 5$ inches beyond the pin. Let the same arrangement be used also at *E*. The pin plates are slotted and attached to the inner sides of the angles so as to reduce the effect of eccentricity.

The net sectional area of the stiff lower chord member which will be made continuous from *a* to *c* (Fig. 124) is 26 square inches on each side of the member. The composition of this section and the full tensile strength of each plate or shape are as follows:

	NET SECTION.	STRESSES.
1 web plate, 18″ × ½″	5.00 sq. ins.	75 000 lbs.
2 angles, 3½″ × 3½″ × ½″	5.50	82 500
1 pin plate, 11″ × ½″	2.50	37 500
1 pin plate, 17″ × ⅝″	5.62	84 300
Total	18.62	

In determining the net section at the pin, two rivet holes are also deducted. (See Fig. 131.) At the end of the pin plates the web takes a stress of 112 500 pounds, and the two angles 82 500 pounds, the rivet holes being deducted from the web section and one from that of each angle. Since the web's share of the stress is just equal to that carried past the pin by the web as well as that of the narrower pin plate, the only stress that has to be transferred to the angles is that from the wider pin plate. The number of rivets connecting the latter to the angles must not be less than 84 300/6610 = 13, while only 6 rivets are needed to connect the narrower pin plate to the web. Although the angles extend past the pin, none of the rivets on the right of the pin should be counted in the 13 required. In order to avoid reducing the net section of the member, the rivets in the tie plates are given the same pitch as those in

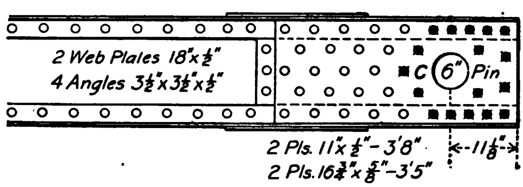


Fig 131.

the vertical legs of the angles until the pin plates are passed. On dividing the stress in proportion to the bearing on the pin, the $\frac{1}{2}''$ web and $\frac{1}{2}''$ pin plate take 60 000 pounds each, while the $\frac{5}{8}''$ pin plate takes 75 000 pounds. This stress due to bearing in the $\frac{1}{2}''$ pin plate exceeds that which it can carry past the pin by $60\,000 - 37\,500 = 22\,500$ pounds, and hence 3 rivets are required on the right of the pin to transfer this excess to the web and to the other pin plate. More than this number are inserted (Fig. 131).

The pin bearing at panel point C in the upper chord is to be designed to take the horizontal component of the full tensile strength of the diagonal Cd, or 199 200 pounds. The linear bearing on each side is $199\,200/(2 \times 5.25 \times 22\,000) = 0.863$ inch, and hence a pin plate of the minimum allowable thickness is required. As the web plate is $\frac{11}{16}$ of an inch thick, the pin plate's share of the bearing is $99\,600 \times 6/17 = 35\,200$ pounds. Since the only change in the section of the upper chord at C is in the web plates, the stress in the pin plate must be transferred to the web plate, and therefore requires $35\,200/6610 = 6$ rivets. Most of these are to be placed on the right-hand side of the pin, but in so small a plate the appearance is improved by making both sides alike, as shown in Fig. 132.

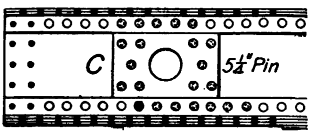


Fig. 132.

At the hip joint B (Fig. 124) the entire stress in the upper chord member BC and that in the end post aB are transferred to the pin, all the plates and shapes except the hinge or lap plates being faced parallel to the bisecting plane of the angle and about $\frac{1}{8}$ inch from it. The hinge plates of each member consist of two plates, located on the inside in one case and on the outside in the other, and extend past the pin. Their purpose is to prevent any accidental blow from displacing these

members, and to facilitate the erection of the truss. The combined pin plates on both members must be arranged with respect to each other so as to provide a clearance of at least $\frac{1}{8}$ inch between them. The full strength of the chord BC is $48.61 \times 13\,580 = 660\,100$ pounds, while that of the end post aB is $53.09 \times 11\,740 = 623\,300$ pounds. The linear bearings required for each side on a 6-inch pin are respectively $2\frac{1}{2}$ and $2\frac{5}{8}$ inches.

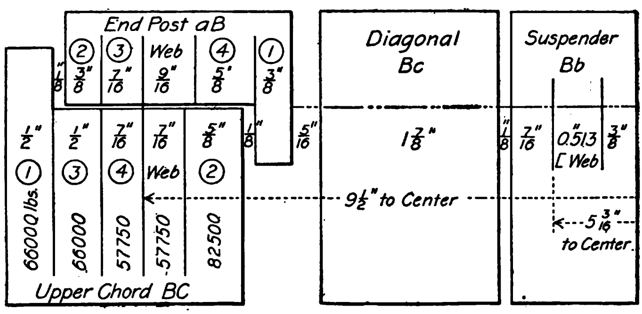


Fig. 133.

Fig. 133 shows the thicknesses and the arrangement of the pin plates, and the bearing stresses which they take. It is to be remembered that the distance back to back of the angles, or out to out of the web plates, is the same in both members. Beyond the pin plates the stresses in the plates and angles composing the chord BC must be directly proportional to their gross areas. Considering only one side of the member, the division of stresses is as follows:

	Gross Areas.		Stresses.
$\frac{1}{2}$ cover plate	5.685		
1 upper angle	2.87	8.555 sq. ins.	116 200 lbs.
1 web plate		7.88	107 000
1 lower angle	2.87		
1 flat	5.00	7.87	106 900
		24.305 sq. ins.	330 100 lbs.

Since nearly all the stresses in the pin plates must be transferred to the angles, the ideal arrangement of pin plates would be to have the same thicknesses on the outside of the vertical legs of the angles as (in symmetrical order) on the inside of the web plates, the plates outside of the angle being either of equal thickness or of regularly decreasing thickness. The plate next to the angle should be the longest and the outside one the shortest, those on the inside of the web being of the same successive lengths to make the entire arrangement symmetrical. Such a plan can be carried out completely in connection with the middle web of a chord having three webs, but with the outside webs it can only be approximated.

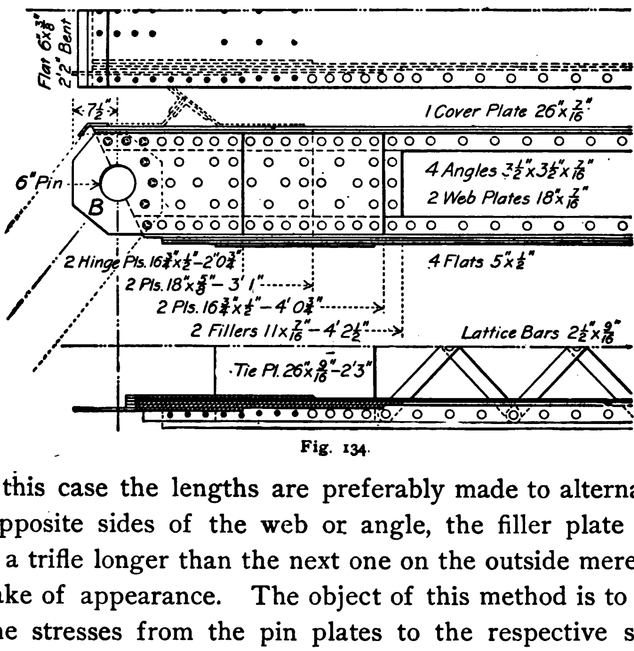


Fig. 134.

In this case the lengths are preferably made to alternate on the opposite sides of the web or angle, the filler plate being made a trifle longer than the next one on the outside merely for the sake of appearance. The object of this method is to transfer the stresses from the pin plates to the respective shapes composing the body of the chord by the most direct route and to put as many of the rivets in double shear as possible. Its application to the chord *BC* is illustrated in Fig. 134.

The web plate takes 107 000 − 57 750 = 49 250 pounds more stress than it gets directly from the pin bearing, and as the $\frac{7}{16}''$ plate, which also serves as a filler between the upper and lower angles, is not directly connected to the angles, it will be assumed that 49 250 pounds of its stress is transmitted directly into the web plate, and that the balance of its stress, or 8500 pounds, is to be transferred to the angles indirectly through all the other plates, including the web, in proportion to their respective thicknesses. The division gives plates 1, 2, and 4 (see Fig. 133) extra stresses of 2100, 2600, and 2100 pounds respectively. The total stress in plate 1 is then 68 100 pounds, and 11 rivets in single shear are required to transfer its stress to the upper and lower angles. Let the length of the plate be extended to include 6 rivets in the shorter angle, since its stress is to be divided about equally between the upper and lower angles. (See Fig. 134.) These 12 rivets also pass through plate 2, and being thus in double shear, their bearing in the angle will determine the stress which they can take out of both plates 1 and 2. This bearing value is 12 × 8430 = 101 200 pounds, while the combined stress in both plates equals 153 200 pounds, leaving a balance of 42 000 pounds to be taken by additional rivets in single shear. The number required is 42 000/6610 = 7. Plate 2 is accordingly extended to engage 4 rivets in each angle beyond the extremity of plate 1.

The combined strength of plates 1, 2, and 3 is 221 300 pounds, while the bearing value of the 20 rivets which are in double shear is 168 600 pounds. The balance requires 52 700/16 610 = 8 rivets in single shear. Plate 3 is therefore extended 4 rivet spaces beyond plate 2. The number of rivets required to carry the stresses from the filler plate 4 to the web and to plate 2 is (49 250 + 1800 + 2600)/6610 = 9. Many more than this number must be inserted to keep the plates in contact and to give the necessary stiffness in compression. It will be noticed that

3 rivets are placed between the angles in the last vertical row in each pin plate, while in the other rows alternate rivets are omitted, the resulting pitch being the maximum allowed. The rivet pitch in the angles is 3 inches, and this pitch is to be continued far enough to satisfy the specification that at the ends of compression members the pitch shall not exceed four times the diameter of the rivets for a length equal to twice the width of the member. Beyond that limit the pitch is increased to a distance somewhat less than 6 inches, depending upon the lacing.

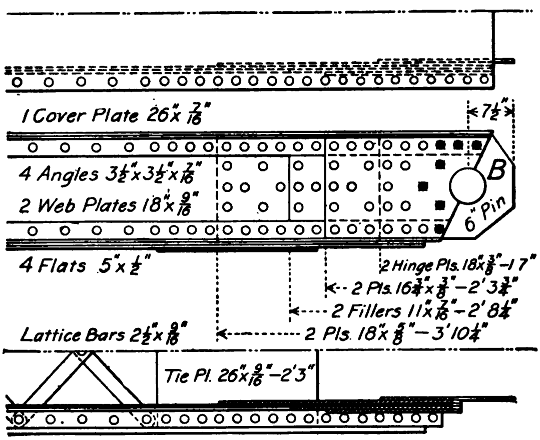


Fig. 135.

In the same manner the lengths of the pin plates on the end post and their riveting may be determined. Fig. 135 shows the result for the upper end of the end post. That for the lower end differs from this only in the effect of the additional vertical load transferred by the end floor beam (see Plate IV). In order to provide for the connection of the portal bracing, the middle row of rivets shown in Fig. 135 will be replaced by a double row of field rivets, the outer rows in the web plate being moved a little closer to the angles.

When a pin plate is shorter than its width, it is desirable to investigate it as a beam with its reactions at the rivet lines of the angles, and the load at the pin. In the case of the hinge plate on the end post it was found that a solid plate 13.6 inches long at its center is required. If the plate did not extend beyond the pin, its length would have to be increased.

In the upper chord and end posts of Fig. 111, Art. 82, the cover plate is 34 inches wide, and in order to avoid an excessive thickness of pin plates on the webs, short intermediate web

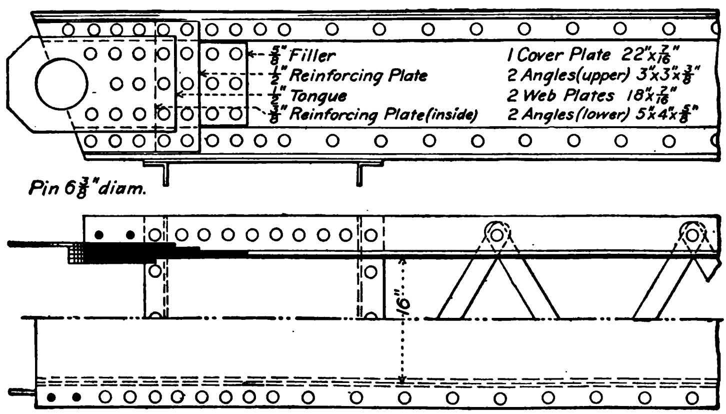


Fig. 136.

plates or diaphragms are inserted which also aid in a more direct distribution of stresses from the pin to the plates and shapes composing the member.

The results of some pin-plate tests are published in a paper by T. H. JOHNSON read before the Engineering Society of Western Pennsylvania, and reprinted from its Transactions in Engineering Record, vol. 28, page 39, June 17, 1893.

Fig. 136 shows an actual example which illustrates a frequent practice of inserting in the pin plates a sufficient number of rivets to carry their respective stresses out of the plates, but

without any regard to where those stresses are to be transmitted. The effect of this arrangement is to stress the web plates near the end of the member far beyond the safe value. It will be observed that only three rivets in the lower angles and four in the upper ones are in double shear, and that the wide ½-inch outer pin plate should be considerably extended to engage additional rivets through the angles, while the inside plate should be made longer than any other one.

Another typical example of an inefficient design, but which gives an appearance of adequate strength, is that in which the filler plate is extended about twice as far as any other pin plate, and contains as many rivets as can be crowded in with a 3-inch pitch in both longitudinal and transverse directions.

Art. 97. Tie Plates and Lacing.

Specification. — At the ends of compression members the pitch of rivets shall not exceed four diameters of the rivet, for a distance equal to twice the greatest width of the member.

All segments of compression members connected by lacing only, shall have tie plates placed as near the ends as practicable. The tie plates shall have a length not less than the greatest width of the member, and a thickness not less than one-fortieth of the distance between the lines of connecting rivets, measured at right angles to the length of the member.

Single lattice bars shall have a thickness of not less than one-fortieth, and double bars connected by a rivet at the intersection of not less than one-sixtieth of the distance between the rivets connecting them to the members; and their width shall be:

For 15″ channels, or built sections with 3½″ or 4″ angles	2½ inches (⅞″ rivets).
For 12″ and 10″ channels, or built sections with 3″ angles	2¼ inches (¾″ rivets).
For 9″ and 8″ channels, or built sections with 2½″ angles	2 inches (⅝″ rivets).

The distance between connections of lattice bars shall not exceed eight times the least width of the segments connected.

In order to determine how close to the end of any member the tie plate — also called stay or batten plates — may be placed, it is necessary to draw the limiting outlines in direction as well as position of all the members which meet at the same panel point. Some clearance must be allowed so as to facilitate erection, and the riveting in the other leg of the angle or in the web of the channel, as the case may be, affects the location of the end rivet in the tie plate. The length of the tie plate must not only comply with the specification, but it is frequently made a little longer than the minimum limit so as to conform to the necessary spacing of the lattice bar connections. Where a tie plate is close to a web diaphragm of a member its length may be reduced. In tension members the tie plates are usually shorter than in compression members.

In members built up so as to require rivets between those connecting the lattice bars to the member, the space between adjacent connections is preferably a multiple of the rivet pitch, the latter not being expressed closer than a full eighth of an inch. In double lacing the multiple may be that of any number whether odd or even, but in single lacing the number should be an even one. In single lacing that on opposite sides of the member is arranged so that if both are projected on a parallel plane, the combined projections are symmetrical about the central axis. The bars generally make an angle with a plane perpendicular to the axis of the member, not to exceed 30 degrees for single lacing nor 45 degrees for double lacing. Some specifications limit the distance between the connections to eight times the least width of the segments connected, or to the width of the channel plus nine inches. The spacing should also be such as to provide adequate openings for painting the interior surface of the member. In members of minor importance or in tension members the angles may slightly exceed these values. WADDELL's specification mentions only single

lacing and prescribes lacing angles to be used when bars exceeding $2\frac{1}{2}'' \times \frac{1}{2}''$ would otherwise be required.

The American Bridge Company's Standards for Structural Details contains a table giving the maximum distances between connecting rivets for different thicknesses of bars, in accordance with the specification at the beginning of this article, the standard form and length of the ends beyond the rivets, and the ordered as well as the finished lengths of the bars. Examples of tie plates and lattice bars in both single and double lacing are shown in Figs. 111 and 134 and Plates IV, V, and VII.

Sometimes where the tie plates of posts cannot be placed very close to its ends, as in the case where the flanges are turned outward and cut off to clear the upper chord, short middle web diaphragms are inserted which extend to within a few inches of the pin. This diaphragm may be composed either of a channel or of a plate and two angles.

ART. 98. END BEARINGS.

SPECIFICATION. — Every span must be provided with some means of longitudinal expansion and contraction due to changes of temperature over a range of one hundred and fifty degrees Fahrenheit. Every span must be anchored at each end to the pier or abutment in such a manner as to prevent the slightest lateral motion, but so as not to interfere with the longitudinal motion of the trusses due to changes of temperature or loading.

The greatest allowable pressure upon expansion rollers of fixed spans, when impact is considered, shall be determined by the equation $p = 600\,d$, where p is the allowable pressure in pounds per linear inch of roller, and d is the diameter of the roller in inches. The least allowable diameter for expansion rollers is four inches. The bearings shall be so designed as to permit a free movement of the rollers in the longitudinal direction of the span sufficient to take up the extreme variations in length due to temperature changes and deflections, and at the same time prevent any transverse motion of the end of the span.

All shoe plates, bed plates, and roller plates are to be so stiffened that the extreme fiber stress under bending, when impact is included, shall not exceed 16 000 pounds per square inch. Bed plates shall be so proportioned that the

pressure upon masonry (including impact) will not exceed 400 pounds per square inch.

Pedestals shall be either of cast steel or built up of plates and shapes. In built pedestals, all bearing surfaces of the base plates and vertical bearing plates must be planed. The vertical plates must be secured to the base by angles having at least two rows of rivets in the vertical legs; and the said vertical plates must bear properly from end to end upon the base. No base plate, vertical plate, or connecting angle shall be less in thickness than three-quarters of an inch. The vertical plates shall be of sufficient height and must contain enough metal and rivets to distribute properly the loads over the bearings or rollers. The bases of all cast-steel pedestals shall be planed, so as to bear properly on the masonry or rollers. All rollers and the faces of base plates in contact therewith are to be planed smooth, so as to furnish perfect contact between rollers and plates throughout their entire length. All pedestals, whether built or cast, must have one or more diaphragms between webs, carried up as high as the general detailing will permit, so as to transmit any transverse horizontal thrust to the base without overstraining the webs by bending in their weakest direction.

The details of expansion bearing are described and illustrated in Arts. 44 and 81, while the design of such bearings was fully outlined in Art. 64 with reference to their use in plate girders. As the same principles apply equally to the design of the end bearings of trusses, the external forces, although larger, acting in the same manner, it seems unnecessary to extend the treatment of this subject. To explain and illustrate the design of all the details of the bearing of the truss under consideration in this chapter would also require more space than can well be spared for the purpose.

Attention should be called, however, to experimental investigations of the stresses in friction rollers. The papers named below give additional references to theoretical investigations and experiments. A Review of Professor Grashof's Investigation of the Carrying Capacity of Rollers and Balls, by CARL G. BARTH, may be found in Proceedings of Engineers' Club of Philadelphia for 1893, vol. 10, page 259. A valuable paper by C. L. CRANDALL and A. MARSTON, entitled Friction Rollers, is

published in Transactions of the American Society of Civil Engineers, vol. 32, page 99, Aug., 1894, with additional discussion on page 270. It contains the results of extended experiments on rollers of cast iron, wrought iron, and steel, and of the study of glass rollers under pressure by means of polarized light. A number of specifications have since been revised in conformity with the conclusion reached in this paper that the safe pressure varies directly as the diameter of the rollers, instead of as the square of the diameter, as implied in most of the specifications then in use. See also, MERRIMAN'S Mechanics of Materials, Art. 107.

Reference may also be made to a note on the design of segmental rollers by F. P. McKIBBEN in Engineering News, vol. 36, page 401, Dec. 17, 1896. A correction of one of the formulas was published on page 433, Dec. 31. The design of segmental rollers is also discussed in the references given in Art. 81.

ART. 99. MINOR DETAILS.

SPECIFICATION. — All plates, angles, and channels used in built members of trusses, must, if practicable, be ordered the full length of the member; otherwise the splices must develop the full strength of the member without any reliance being placed on the abutting ends for carrying compression. But in total splices at the ends of sections, perfect abutting of the dressed ends is to be relied upon. However, the splice plates even there must be of ample size and strength for both rigidity and continuity.

As shown in Fig. 111, and on Plates III and V, the upper chord is spliced a short distance to the left of a panel point in the left half of the truss. As the erection of the trusses begins usually with the middle panel, the chord is not spliced within the limits of that panel. In small trusses a splice is generally located in every panel except the middle one, but in larger trusses where the upper chord is horizontal it is often built in parts which are continuous over two panels. Such an arrange-

ment is shown on Plate IV, the web plates being spliced in the shop as the plates in the adjacent panels differ in thickness. The angles and cover plate are continuous, as their section is the same in both panels.

A splice plate is placed on both sides of each web, the outer one extending between the vertical legs of the upper and lower angles. Another plate is put on top of the cover plate, while the tie plate acts also as a splice plate below. All of these plates should be wide enough (longitudinally with respect to the chord) to permit two rows of rivets on each side of the joint. When the chord is large the size is increased, as shown on Plate V. In this case there are four webs, but as only the two outer ones can be spliced, one plate is placed on the inside and two on the outside of each of these two webs.

For the truss whose design is under consideration the splice plates on the side will be made 12 inches long, the joint being placed midway between consecutive rivets of 3-inch pitch in the vertical legs of the angles. The plate on the cover must therefore be 15 inches wide in order to have two rows of rivets on each side of the joint. The tie plate will have its ordinary length as stated in Art. 97. The elevation of one end of a chord member next to the splice was introduced for the sake of illustration in Fig. 132, Art. 96. The field rivets in the top and bottom beyond the first two in each line are for the plates connecting the laterals to the chord. As the chord is preferably built continuous from B to a point near D (Fig. 124), the web only being spliced in the shop, the chord joint shown in Fig. 132 really belongs to D, whose pin plates are made the same as those designed for C. Since the web plates are $\frac{7}{16}''$ thick in BC and $\frac{11}{16}''$ in CD, a filler $\frac{1}{4}''$ thick is required on one side of the web joint near C on the inside of each of the thinner web plates.

When the upper chord of a truss changes its direction in any two adjacent panels, the splice must necessarily be located directly at the pin, and the pin plates should then be designed to transmit the entire stress through the pin in the same manner as for the hip joint, no direct contact being allowed between the adjacent chord members. An important reference to this topic may be found in Proceedings of the Engineers' Club of Philadelphia, vol. 14, pages 155 and 164.

If possible, the stiff lower chord *ac* should have its sides parallel so as to simplify the construction by making all the lattice bars of equal length. It is found that the outside of its web plate is only three-sixteenths of an inch farther from the central plane of the truss than that of the end post. This distance must be increased three-quarters of an inch in order to permit the former web plate to pass the $\frac{3}{8}$-inch pin plate outside of the angles of the end post. At this end of the end post the outside pin plates will be made the hinge plates, so that the hinge plates of the pedestals will be on the inside. An inspection of the bending moment diagram of the pin at *c* shows that this change will slightly reduce the bending moment in the pin, provided the washer to fill the extra space on each side of the central plane is placed directly inside of the web plate, thus leaving unchanged the distances of all the eye-bars from the center except those of the two outer bars in the panel *cd*. Any other position of the washers would materially increase the bending moment and consequently change its diameter.

The diaphragm web between the channels of each intermediate post opposite the floor beam is designed to carry half of the floor beam reaction to the outer channel. The smallest angles allowed for $\frac{7}{8}$-inch rivets, namely, $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ angles, and four in number, together with a web plate $9'' \times \frac{3}{8}''$, will furnish more than sufficient strength for this purpose. The length

of the diaphragm equals the total height of the angles connecting the floor beam to the post.

One of the diagonals of the middle panels must be cut so as to pass the other one, the two parts being connected by a plate on each side, the two plates having the necessary net section and number of rivets in single shear to transmit the full strength of the member. A short tie plate is inserted on each side of the splice. In a similar manner splices are to be designed for one of the upper laterals in each panel. The splice in the lower laterals consists of a single plate in each case. The computations required for these three sets of details are so simple that they will not be given.

ART. 100. CAMBER.

SPECIFICATION. — All trusses must be provided with such a camber that with the heaviest live load on the span, the total camber shall never be quite taken out by deflection. With parallel chords sufficient camber will be obtained by making the panel of the top chord longer than those of the bottom chord by one-eighth of an inch for each ten feet of length. The increased length of the top chord shall be neglected in figuring the lengths of main ties, but shall be considered when figuring the lengths of inclined end posts and counter-braces. Half the increase in length shall be considered in figuring the length of top laterals. One-half of the camber after a span is swung is to be taken out of the track by notching the ties, unless this would cut too deeply into the timber.

The application of this specification to the truss under consideration makes the actual or shop lengths of the upper chord panels $\frac{5}{16}$ inch longer than the nominal length of 25 feet. The length of the main diagonal, without allowance for the clearance of the pins in the pin holes, is 39 feet $10\frac{1}{32}$ inches. The length of the end post and counter-braces is $\frac{1}{16}$ inch longer without a similar allowance. To the several lengths just given $\frac{1}{32}$ inch is to be added for pin clearance in the case of compression members, while it is to be subtracted in the case of tension members.

A more precise method of providing for camber, and which must be applied to larger spans according to the above specification, consists in making the shop length of each tension member shorter than its nominal length by an amount equal to the elongation caused by its stress under the dead and full load when increased by a small percentage. The length of a compression member is correspondingly increased. The same allowance for the pin clearance is to be made as that noted in the preceding paragraph. When the elongation or shortening is computed for the dead and full live load stresses only, the camber is entirely taken out under the full live load. The lengths of secondary members, like the short diagonals in a Baltimore truss, are sometimes made the mean of the nominal length and that obtained in the manner just described.

The deflections at the various panel points are most conveniently found by the graphic method explained in Chap. VII, Part II. The results may be checked either by a separate diagram or by computing the deflection at the middle panel point of the loaded chord, by the method given in Chap. V, Part I.

In an article entitled Camber of Bridges, in Railroad Gazette, vol. 22, page 665, Sept. 26, 1890, THEODORE COOPER states the object of camber and describes its relation to the track surface. See also Camber in Bridge Trusses, by G. H. PEGRAM, in Engineering News, vol. 18, page 21, July 9, 1887.

Special attention was given in the design of the Delaware river bridge to its deflection and camber. The lengths of the members were so arranged that the center of the bottom chord should be about as far below its normal position under a full load on both tracks as when the span is unloaded, it being assumed that the greatest live load on the bridge would very rarely exceed that of a full load on one track. A description of the methods employed to secure this result, to avoid excessive

tension in the stringer connections due to the elongation of the lower chord, and to reduce the effect of secondary stresses due to the subdivision of the panels by the secondary web system, is given by PAUL L. WÖLFEL, in Proceedings of the Engineers' Club of Philadelphia, vol. 14 (1897), page 156.

ART. 101. ANALYSIS OF WEIGHT.

SPECIFICATION. — If in any bridge design the dead load assumed shall differ from that computed from the diagram of sections and the detail drawings by an amount exceeding one percent of the sum of the equivalent live load and actual dead load, the calculations of stresses, etc., are to be made over with a new assumed dead load.

After computing the weight of every member the results for one truss, exclusive of the pedestals, may be classified as follows:

TRUSS MEMBERS.	POUNDS.	ONE-HALF LATERAL AND TRANSVERSE BRACING.	POUNDS.
Intermediate posts	11 632	Upper laterals and connections	5 922
Suspenders	5 406		
Diagonals	26 162	Lateral struts and sway bracing	2 686
Lower chord	29 924		
Upper chord	28 682	Portal bracing	3 604
End posts	18 134	Lower laterals and connections	7 410
Pins	2 632		
	122 572		19 622

If the pins be included with the chords, the weight of the web members is found to be 61 334 pounds, and that of the chords to be 61 238 pounds, thus indicating that the depth chosen is the one which makes the weight of the truss a minimum.

The total weight of these members is made up of the following items:

	POUNDS.	PERCENT.
Main shapes and plates composing members	109 768	77.2
Pin plates	7 608	5.3
Tie plates and lacing	10 676	7.5
Connections, splices, and other details	10 460	7.4
Rivet heads	3 682	2.6
	142 194	100.0

In the first edition of Part III a corresponding analysis was given for a single-track Pratt truss bridge with a span of 142 feet, and designed for a load but little more than half that specified in Art. 83. The corresponding percentages were 76.7, 5.3, 7.1, 6.1, and 4.8. This shows that the relative combined weight of the details is only slightly affected by considerable changes in the loading and specifications.

Since the net weights of the shapes and plates composing some of the members cannot be computed with precision until many of the details are designed, it is desirable to compare the total weights of the several classes of members with their theoretic weights obtained by means of the adopted gross sectional areas and their lengths, center to center of pins, no deduction being made for pin holes. Such a comparison is made in the following table:

TRUSS MEMBERS.	FINAL WEIGHTS.	THEORETIC WEIGHTS.	RATIO.
Intermediate posts and suspender	17 038	11 776	1.447
Diagonals and stiff chord *ac*	33 028	23 352	1.414
Upper chord and end posts	46 816	37 334	1.254
Eye-bars	23 058	20 026	1.151
	119 940	92 488	1.297

These ratios vary somewhat for various types of pin trusses and for different spans, but the difference is comparatively

small. For instance, in one of the fixed spans of the Delaware river bridge, whose length is 533 feet, the upper chord being curved and the panels subdivided (see Fig. 10), the ratios are as follows: Intermediate posts and long suspenders, 1.457; upper chord and end posts, 1.204; eye-bars, 1.137; sub-verticals, 1.975; intermediate horizontal stays or rails to support the long posts, 1.839; and total, 1.228. These values are given by F. C. KUNZ in the article to which reference is made at the end of Art. 82.

By means of such ratios the dead load assumed in computing the stresses may be corrected as soon as the sections of the members are designed, thereby avoiding any revision after the details are designed. No revision was made in this chapter in accordance with this method in order to furnish the data for an example to the student, who should make the revision and observe the consequent effect upon the sections of the members.

The dead load for one span, exclusive of the pedestals, is divided as follows:

	POUNDS.	PERCENT.
Track	77 000	16.4
Steel floor system	107 240	22.9
Trusses and connecting bracing	284 390	60.7
	468 630	100.0

The steel floor system is made up of two end floor beams, each weighing 3537 pounds, four brackets outside of the end floor beams and in line with the stringers, each weighing 303 pounds, together with fourteen stringers and six intermediate floor beams whose weights are given in Arts. 85 and 86.

The excess of the actual dead panel load per truss over that assumed is 33 470 − 30 000 = 3470 pounds. The sum of the equivalent live load and the actual dead load per panel is 79 100 + 33 470 = 112 570 pounds. The excess of the dead panel load is 3.08 percent of this sum, and hence a revision is required according to the specification printed at the head of this article. It will be found that only a part of the sections of members need to be increased on account of their excess of area over that previously required.

ART. 102. GENERAL DRAWING.

Instead of reproducing the general drawing of the truss and its connecting bracing, whose design is given in this chapter, there is inserted in Art. 82 a similar drawing of a single-track through truss bridge of a span of 200 feet, it being a part of one of the standard plans of the Northern Pacific Railway prepared by RALPH MODJESKI (see Plate IV). Four other sheets, not reproduced, belong to the complete set.

The dead load is assumed to be 2700 pounds per linear foot of bridge, one-third of it being concentrated at the upper panel points. The live load consists of two 146-ton locomotives with a combined length of 124 feet, preceded and followed by a uniform load of 4000 pounds per linear foot of track. The top lateral bracing is designed for a wind load of 250 pounds per linear foot, and the bottom lateral bracing for a steady load of 200 pounds and a moving load of 300 pounds per linear foot.

The percentages of impact are as follows: 21 percent for the bending moments of the stringers and end floor beams; 17 percent for those of the intermediate floor beams; 35 percent for the shears of stringers and all floor beams; $7\frac{1}{2}$ percent for the end posts, top and bottom chords, the diagonal Bc and the abutments; 11 percent for the post Cc and diagonal Cd; 14 per-

cent for the post Dd and diagonal De; 30 percent for the post Ee and diagonal Ed.

The stringer (not shown on Plate IV except in section) has a web plate $36'' \times \frac{7}{16}''$, flanges composed of two angles $6'' \times 6'' \times \frac{5}{8}''$, five pairs of stiffeners $3\frac{1}{2}'' \times 3'' \times \frac{3}{8}''$, all being crimped over the flange angles except the inner one of the middle pair, to which a cross-frame is connected. The connections at each end consist of two angles $6'' \times 6'' \times \frac{1}{2}''$ and two fillers $9'' \times \frac{5}{8}''$. The laterals are composed of single angles $4'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$, arranged like the web members of a Warren truss, in a continuous series from end to end of the span, there being three panels of the lateral bracing in each panel of the bridge. The cross-frame has an angle $4'' \times 3'' \times \frac{3}{8}''$ for each horizontal brace and an angle $3'' \times 3'' \times \frac{3}{8}''$ for each diagonal. The bracket in each line of stringers outside of the end floor beam projects 15 inches beyond the center of the floor beam.

All the material in the span, except where otherwise noted, is medium steel. All rivets are of soft steel and $\frac{7}{8}$ inch diameter. The first figure in the size of any angle marked on the drawing indicates the width of the leg shown.

The student's attention is called to the following features which differ from those indicated in Figs. 113 to 135 inclusive. The construction of the end of the intermediate floor beam and its clearance for the eye-bars, the wide spacing of the stringers; the use of eye-bars for all the diagonals; the larger diameter of the pin at e; the addition of a collision strut which in turn necessitates a pin connection at panel point b; the use of extra angles at the bottom of the upper chord and end post to balance the section, the lattice bars being connected to the inner ones; the position of the eye-bars of Bc on the outside of the upper chord, thus reducing the width of the chord; the use of adjustable upper laterals; the combined pin and splice plates on the sides of the upper chord; the combined lateral connecting plate

and splice plate on top of the chord, the connection of the posts at *d* and *e* to the lateral connecting plates by means of U-plates; and the connection of the lateral plates at *a*, *b*, and *c* to the stiff lower chord. Some other items were referred to in Chap. VIII.

The student should carefully study the details of other modern designs from the blue prints in the college collection, or by visits of inspection to actual bridges in the vicinity, and record in his note book the special features of the construction in each case. If this is done in some regular order, many points will be noticed that otherwise would be overlooked. The study of shop drawings on which each member is shown separately in the manner described in Art. 17 is especially important with reference to the location of rivets, their relation to center lines and to points of intersection of axes of connecting members, and their influence on the exact lengths of the projecting ends of members. They also show modifications in spacing to avoid interference, and what rivets are flattened or countersunk to secure the necessary clearance. These are frequently not shown on general drawings.

A complete set of drawings for a truss like the one treated in this chapter includes the following sheets. The stress sheet, general drawing, end post and part of upper chord, balance of upper chord and stiff lower chord, suspender and intermediate posts, stiff diagonals (if any), portal and sway bracing, upper and lower lateral bracing, end bearings, and the erection diagram. The general drawing is sometimes omitted.

ART. 103. BRIDGE DESIGN REFERENCES.

In Art. 7 references are given to the principal literature on bridge design. The following articles which have appeared in the engineering periodicals will also repay careful reading. The reference to WADDELL's elaborate paper is here repeated in order to notice the abstracts of the paper and its discussion.

Bridge Design. By H. J. LEWIS. Engineering News, vol. 26, page 367, Oct. 17, 1891.

Bridge Details. By E. SWENSSON. Railroad Gazette, vol. 24, page 156, Feb. 26, 1892.

Some Disputed Points in Railway Bridge Designing. By J. A. L. WADDELL. Transactions American Society of Civil Engineers, vol. 26, pages 77–282, Feb., Mar., 1892. An abstract of the paper is given in Engineering News, vol. 26, pages 563, 610, 621, Dec. 12, 26, 1901. See also editorial on page 566. An abstract of the paper and of the discussion is also given in Railroad Gazette, vol. 24, pages 772, 779, 811, and 823, Oct. 14, 21, 28, Nov. 4, 1892.

Notes on the Designing of Metallic Structures. By O. J. MARSTRAND. Engineering Record, vol. 29, page 187, Feb. 17, 1894.

Advance in the Design of Bridge Superstructure. By G. S. MORISON. Engineering News, vol. 30, page 80, July 27, 1893.

Details of Construction of Engineering Structures. By C. C. SCHNEIDER. Engineering Record, vol. 32, pages 256, 364, 382, Sept. 7, Oct. 19, 26, 1895.

Some Hints on Bridge Designing. By OSCAR SANNE. Journal of Western Society of Engineers, vol. 4, page 229, April, 1899.

Exhibit of Typical American Bridges at the Paris Exposition. By G. L. FOWLER. Engineering News, vol. 44, page 10, July 5, 1900.

The Development of the Nineteenth Century in Bridge Design and Construction. Editorial. Engineering News, vol. 44, page 405, Dec. 13, 1900.

Excessive Refinement in Bridge Design. Editorial. Engineering Record, vol. 44, page 393, Oct. 26, 1901.

CHAPTER X.

DESIGN AND DETAILING OF A HIGHWAY BRIDGE.[1]

ART. 104. DATA OF THE DESIGN.

It is proposed to design a highway bridge of 140 feet span which shall, in addition to highway traffic, carry that of trolley cars. The roadway is to be 18 feet in width from center to center of trusses. The trolley track is to be on one side of the roadway and its center 5 feet clear of the truss. On the opposite side of the roadway, on the outer side of the truss, will be a foot-walk 5 feet wide, supported on cantilever brackets attached to the posts. Judging from the rapid increase in trolley wheel loads during the past few years, it is advisable to construct all trolley bridges of such strength that they shall be sufficient to carry the traffic of heavy interurban cars. This bridge will accordingly be designed and detailed in accordance with Class B of COOPER's General Specifications for Steel Highway and Electric Railway Bridges and Viaducts (edition of 1901), with two exceptions as follows:

First, omit § 3 and § 63; second, insert these clauses in § 48: 'When the wind-load stress is taken into account, together with live-load stress in any truss member, two-thirds of it shall be considered as live-load equivalent and is to be added to the live-load stress in computing the total stress.' 'When a post is fixed at its ends the flexural stress caused by the wind shall be computed by considering the ends fixed, but in computing the total stress due to combined loads it shall be considered hinged at both ends.

On account of the uniformity of chord stresses and the small web stresses in the Bowstring truss, this form will be used.

[1] By F. O. DUFOUR, C.E., Instructor in Civil Engineering in Lehigh University.

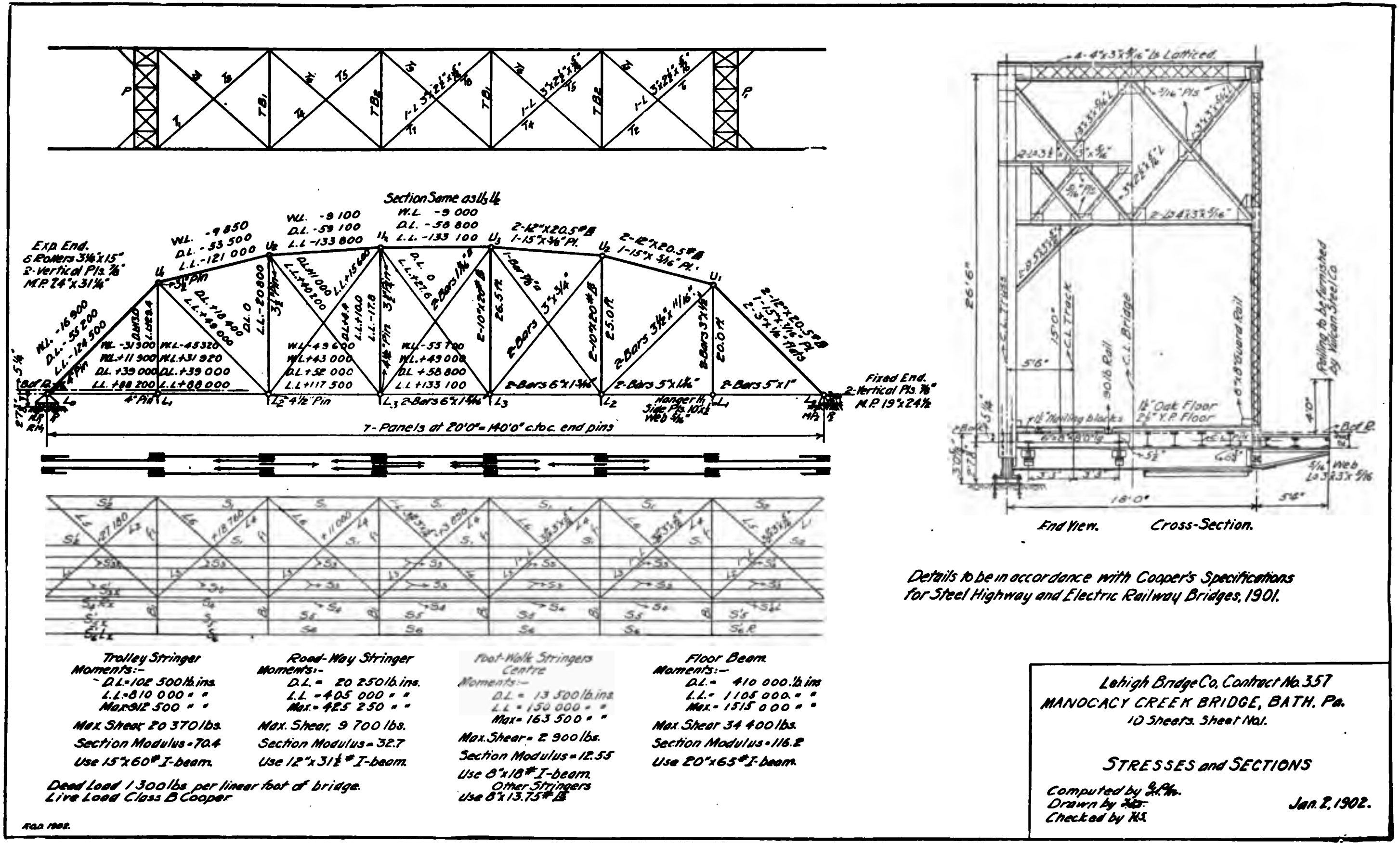

Section Same as U_3L_4
Exp. End.
6 Rollers 3¼"x15"
2-Vertical Pls. ⅞"
M.P. 24"x31½"
Fixed End.
2-Vertical Pls. ⅞"
M.P. 19"x24½"
7-Panels at 20'0" = 140'0" c.to c. end pins
End View.
Cross-Section.
Railing to be furnished by Vulcan Steel Co.
Details to be in accordance with Cooper's Specifications for Steel Highway and Electric Railway Bridges, 1901.
Trolley Stringer
Moments:-
D.L.=102 500 lb.ins.
L.L.=810 000 " "
Max.=912 500 " "
Max. Shear 20 370 lbs.
Section Modulus = 70.4
Use 15"x60# I-beam.
Road-Way Stringer
Moments:-
D.L.= 20 250 lb.ins.
L.L.= 405 000 " "
Max.= 425 250 " "
Max. Shear 9 700 lbs.
Section Modulus = 32.7
Use 12"x31½# I-beam.
Foot-Walk Stringers Centre
Moments:-
D.L.= 13 500 lb.ins.
L.L.= 150 000 " "
Max.= 163 500 " "
Max. Shear = 2 900 lbs.
Section Modulus = 12.55
Use 8"x18# I-beam.
Other Stringers
Use 8"x13.75# I-beam
Floor Beam
Moments:-
D.L.= 410 000 lb.ins.
L.L.= 1 105 000 " "
Max.= 1 515 000 " "
Max. Shear 34 400 lbs.
Section Modulus = 116.2
Use 20"x65# I-beam.
Dead Load 1 300 lbs. per linear foot of bridge.
Live Load Class B Cooper
Lehigh Bridge Co. Contract No. 357
MANOCACY CREEK BRIDGE, BATH, Pa.
10 Sheets. Sheet No. 1.
STRESSES and SECTIONS
Computed by
Drawn by
Checked by
Jan. 2, 1902.

Fig. 137.

This truss has its upper chord pins approximately on the arc of a parabola which passes through the end pins and the middle top panel points. The general dimensions of the truss are as given on the stress sheet (Fig. 137).

The dead load per linear foot is estimated as 1300 pounds. This estimate is based upon the weight of a similar bridge which has been erected, and it is more likely to be correct than any value derived from empirical formulas. The live load for the truss is taken from table A (§ 38, Cooper). The panel length will be 20 feet, thus making 7 panels per truss. The dead panel load is $(1300/2)20 = 13\,000$ pounds. Taking the trolley track load as covering 8 feet of roadway, the roadway load as covering the remaining 10 feet, and the foot-walk as loaded with the roadway load, the live panel load for each truss will be 29 400 pounds. The dead, live, and wind-load stresses are now computed by the methods of Part I, and recorded on a stress sheet (Fig. 137).

A careful consideration of the entire set of specifications should now be made, as much misunderstanding in the design will thus be avoided. The spacing of stringers and the general arrangement of the floor system should be decided upon and placed on the stress sheet.

ART. 105. STRINGERS.

TROLLEY STRINGERS.—The dead load on these stringers is the weight of the 4-inch floor plus the weight of the ties and the rails. The rail is assumed to weigh 30 pounds per linear foot. The oak ties, 6 × 8 inches and 8 feet long, weigh $48/12\,(8 \times 4\frac{1}{2}) = 144$ pounds each, and if they are spaced 14 inches from center to center, there are 17.14 ties to a panel, or $(17.14 \times 144)/(2 \times 20) = 62$ pounds per linear foot. Considering the stringer nearest to the truss, and estimating the width of a post

as 12 inches, the floor plank will weigh $4\frac{1}{2} \times 4 \times 5 = 90$ pounds per linear foot. The total dead load per linear foot is hence $30 + 62 + 90 = 182$ pounds, and the maximum bending moment due to this, since the length of the stringer is 20 feet, is

$$\tfrac{1}{8}(182 \times 20 \times 20 \times 12) = 109\,200 \text{ pound-inches.}$$

In order to obtain the maximum live-load moment, the two loads of 12 000 pounds each should be so placed that the center of the stringers is midway between the center of gravity of the loads and one load (Part I, Art. 91).

From this rule it is found that this load comes at $7\frac{1}{2}$ feet from the end. The maximum bending moment occurs under this load, and its value is,

$$\tfrac{1}{20} \times 12\,000\,(2\tfrac{1}{2} + 12\tfrac{1}{2})\,7\tfrac{1}{2} \times 12 = 810\,000 \text{ pound-inches.}$$

The dead-load moment under this wheel is, $(182 \times 20 \times 7.5 - 7.5^2 \times 182)\,12/2 = 102\,500$ pound-inches. The total moment is the sum of the dead and live load moments, or 912 500 pound-inches. Then $912\,500/13\,000 = 70.4$ inches^3 is the section modulus necessary. A 15-inch 60-pound I-beam has a section modulus of 81.2 inches^3, and is hence stronger than required, but it will be used. The maximum shear is readily seen to be $12\,000(1 + \frac{1}{2}) + (182 + 55)\,20/2 = 20\,370$ pounds. The moment due to the weight of the beam itself is, $(55 \times 20 \times 20 \times 12)/8 = 33\,000$ pound-inches, but as this is less than $\frac{1}{10}\,(912\,500)$ it need not be considered (§ 55, Cooper).

ROADWAY STRINGERS. — The dead load for these stringers consists of only the weight of floor covering, which is $2 \times 4 \times 4\frac{1}{2} = 36$ pounds per linear foot. The bending moment due to this weight at $7\frac{1}{2}$ feet from the end is $\frac{1}{2}\,(36 \times 20 \times 90 - 36 \times 7\frac{1}{2}) = 20\,250$ pound-inches. This is the dead-load moment under the wheel that causes the maximum live-load moment. Here

the loads are placed in the same position as for the trolley stringer. The concentration being 6000 pounds,

$\frac{6000}{20}(2.5 + 12.5) \times 7.5 \times 12 = 405\,000$ pound-inches is the maximum live-load moment. The total bending moment is 20 250 + 405 000 = 425 300 pound-inches, which requires a section modulus of 425 300/13 000 = 32.7 inches^3. A 12-inch $31\frac{1}{2}$-pound I-beam satisfies this condition and will be used. The maximum shear is $6000\ (1 + \frac{1}{2}) + (36 + 31.5)\ 20/2 = 9700$ pounds. The moment in the beam due to its own weight is $\frac{1}{8}(31.5 \times 20 \times 20 \times 12) = 18\,900$ pound-inches, but as this is less than $\frac{1}{10}(425\,300)$, it need not be considered.

FOOT-WALK STRINGERS. — The bending moment due to the uniform load of 100 pounds per square foot is, for the center stringer,

$$\tfrac{1}{8}(2.5 \times 100 \times 20 \times 20 \times 12) = 150\,000 \text{ pound-inches.}$$

The moment due to dead load, which consists of the weight of flooring only, is $\frac{1}{8}(2.5 \times 2 \times 4\frac{1}{2} \times 20 \times 20 \times 12) = 13\,500$ pound-inches. The maximum moment is the sum of these, or 163 500 pound-inches, and then 163 500/13 000 = 12.55 inches^3 is the section modulus required. An 8-inch 18-pound I-beam will be used, as it satisfies the conditions (§ 57, Cooper). The moment due to the weight of the beam itself is $\frac{1}{8}(18 \times 20 \times 20 \times 12) = 10\,800$ pound-inches, and as this is less than $\frac{1}{10}(163\,500)$, it can be neglected.

The outer and inner stringers of the foot-walk will consist of 8-inch channels. The inner one must resist one-half the above moment, while the outer one will, in addition to this, sustain a railing estimated to weigh 60 pounds per linear foot. The railing will be connected to the stringers at the ends and at the one-third points. Both inner and outer stringer will be made the same on account of economy of construction. The moment

due to the railing is $\frac{1}{8}(20 \times 60 \times 3 \times 20 \times 12) = 32\,000$ pound-inches. The total moment which the outer stringer must stand is $32\,000 + 163\,500/2 = 113\,750$ pound-inches, and this requires a section modulus of $113\,750/13\,000 = 8.75$ inches^3. An 8-inch 13.75-pound channel must be used (§ 57, Cooper). The moment due to the weight of the stringer may be neglected. The maximum shear for the inner stringer is, $\frac{1}{2}(13.75 \times 20 \times 2.5 \times 100 \times 20/2 + 2.5 \times 2 \times 4\frac{1}{2} \times 20/2) = 1499$ pounds. In the same manner the maximum shear for the middle stringer is found to be 2905 pounds, and for the outer stringer, railing included, 2100 pounds.

The masonry plates for the end stringers can now be computed (§ 126 and § 130, Cooper). They are as follows: For the trolley stringer, area $= 20\,370/250 = 81.8$ square inches, and length $= 81.8/6 = 13.63$ inches, since the width of flange of trolley stringer is 6 inches; in like manner for the roadway stringer the plate must be 5×7.8 inches, for the center foot-walk stringer 2.35×3.6 inches, and for the channel stringers 4×3 inches. These dimensions should not be taken as final, as in all probability they will be changed in order to secure good details. The thickness of all plates should be one-half inch.

ART. 106. FLOOR BEAMS.

The live load must be so placed on the stringers that the sum of the reactions for two adjacent stringers shall be a maximum. The trolley live load will be assumed to be the only live load acting. The dead-load concentration under the first trolley stringer is as follows:

Due to flooring, $5 \times 4 \times 4\frac{1}{2} \times 20$	= 1800 pounds.
Due to trolley stringer, 60×20	= 1200 pounds.
Due to trolley rail, 30×20	= 600 pounds.
Due to cross-ties, 62×20	= 1240 pounds.
Total for trolley stringer	= 4840 pounds.

The concentration under second trolley stringer is slightly less, but will be considered the same. The dead-load concentration under a roadway stringer is:

Due to flooring, $2 \times 4 \times 4\frac{1}{2} \times 20 =$ 720 pounds.
Due to stringer, 31.5×20 $=$ 630 pounds.

Total for roadway stringer = 1350 pounds.

All roadway concentrations are considered equal. The left reaction, due to these concentrations, is:

$$\tfrac{1}{17}\left[4840\,(8.75+15.25)+1350\,(0.75+2.75+4.75+6.75)\right]$$

= 8020 pounds, and the dead-load bending moment for the floor beam is $8020 \times 8.25 \times 12 - 4840 \times 6.5 \times 12 = 416\,460$ pound-inches. The maximum live-load concentration is 18 000 pounds, and the left reaction due to this is $(2 \times 18\,000)/17 = 25\,400$ pounds. The width of posts being assumed as 12 inches, the length of the floor beam is 17 feet. The live-load moment is $(25\,400 \times 8.25 - 18\,000 \times 6.5)\,12 = 1\,105\,000$ pound-inches. The maximum bending moment now is $1\,105\,000 + 416\,460 = 1\,521\,460$ pound-inches, which requires a section modulus of $1\,521\,460/13\,000 = 117$ inches^3. A 20-inch 65-pound I-beam will be used. The moment due to the weight of the beam itself is $\frac{1}{8}(65 \times 20 \times 20 \times 12) = 39\,000$ pound-inches, which can be neglected, as it is less than $1\,521\,460/10$ (§ 55, Cooper). The maximum shear is $(25\,400 + 7900 + 17 \times 65) = 34\,400$ pounds. The reaction at the other end will be less, but the same connections will be used at each end, as at some future date the track may be changed.

FOOT-WALK BRACKET. — This is the floor beam for the sidewalk. The concentrations are (see Art. 105 under foot-walk stringers) as follows:

At inner stringer, $2 \times 1500 = 3000$ pounds.
At center stringer, $2 \times 2900 = 5800$ pounds.
At outer stringer, $2 \times 2100 = 4200$ pounds.

The bending moment at the post is $(5800 \times 2\frac{1}{2} + 4200 \times 5)12 = 426\,000$ pound-inches. By reference to the stress sheet it will be seen that the top of bracket is $19\frac{1}{8}$ inches from the bottom of the post. Assuming that the center of gravity of the angle to be used is $\frac{1}{2}$ inch from its back, and such assumption is near enough for this computation, the effective depth of the bracket at the post is $18\frac{1}{2}$ inches, or say 18 inches. Then the stress in the top flange is $426\,000/18 = 23\,700$ pounds, and this demands a net area of $23\,700/13\,000 = 1.82$ square inches. One angle will then require 0.91 square inch net area. It is specified that two $\frac{7}{8}$-inch rivet holes shall be deducted from the angle section. A $3 \times 3 \times \frac{5}{16}$ inch angle will be used, this having a net area of $1.78 - (0.875 + 0.125)0.625 = 1.15$ square inches, which is greatly in excess of the required area, but the smallest obtainable satisfying the conditions. The stress in the bottom flange is slightly less, but the same angles will be used as for the top flange. A solid $\frac{5}{16}$-inch web will be used throughout. The connection to the post will be made by the detail shown on the stress sheet.

ART. 107. TENSION MEMBERS.

The eye-bars should not be greater in width than four-thirds the diameter of the pin to which they are attached (§ 104, Cooper), and in general the thickness of a bar should not be less than one-sixth its width. Pins safe in bending are liable to be deficient in bearing. For this reason it is advisable to design the bars so that they will not be deficient in bearing on pins of minimum diameter. A relation satisfying this condition will now be deduced.

Let t be the thickness of the bar, W its width, P the total stress it is required to sustain, D the diameter of the smallest pin, and S the allowable unit bearing stress. Then $SDt = P$. But $D = \frac{3}{4}W$, and $W = 6t$. Substituting these values and re-

ducing, $9\,St = 2\,P$. Here $S = 18\,000$ pounds per square inch, and hence $t = 0.00352\,P^{\frac{1}{2}}$ is the minimum allowable thickness of a bar. Guided by this, and knowing that bars under six inches should be ordered in variations of one-half inch and bars over six inches should be ordered in variations of one inch, it is now easy to find the sizes of the eye-bars whose maximum stress is known. The maximum stress in the web members is readily found by adding one-half of the dead-load stress to the live-load stress (§ 45, Cooper). This sum divided by the number of bars which are to carry it, gives the load P above. The minimum thickness is then computed. Next the area and the maximum width are found, and lastly the final size. For web members, the widths should generally decrease from the ends toward the middle of the truss. For lower chord members the widths should generally increase from the ends toward the middle. According to § 52 of the Specifications the wind stresses must be considered in designing these sections.

The maximum stress in L_0L_1 is,

$$88\,200 + 39\,000 + 31\,900 \times 0.8 = 152\,720 \text{ pounds.}$$

The maximum stress in L_1L_2 is,

$$88\,200 + 39\,000 + 45\,320 \times 0.8 = 163\,450 \text{ pounds.}$$

The maximum stress in L_2L_3 is,

$$117\,500 + 52\,000 + 43\,000 = 212\,500 \text{ pounds.}$$

The maximum stress in L_3L_3 is,

$$133\,100 + 58\,000 + 49\,000 = 240\,900 \text{ pounds.}$$

In computing these stresses eight-tenths of the negative wind load is added only when it is greater than the positive wind load (§ 55, Cooper).

The number of bars to be taken for any member is a matter of choice in some respects. An even number should, of course, always be taken, except when only one is needed. They should be so chosen and packed that the flexure of the pin is a minimum. A large number decreases this flexure while the reverse increases it. It costs almost the same to forge a large eye-bar as it does a small one, while the manufacture of large pins is much more costly than the manufacture of small ones.

The problem resolves itself into this form, namely, that the cost of eye-bars and pins shall be a minimum. The shop practice of different plants modifies the results obtained as to the number of bars which satisfy these conditions, and therefore no hard and fast rule can be given. The stress in the heaviest eye-bar due to the weight of the bar itself is readily computed to be 2100 pounds per square inch, which need not be considered. A table can now be formed as follows:

MEMBER.	P POUNDS.	NUMBER OF BARS.	t INCHES.	UNIT STRESS. POUNDS/SQ.IN.	AREA REQUIRED IN SQ. INCHES.	MAXIMUM WIDTH. INCHES.	BARS USED. INCHES.
L_0L_1	76 360	2 eye	0.88*	15 625	4.90	5.57	5×1
L_1L_2	81 730	2 eye	0.91*	15 625	5.23	5.75	$5 \times 1\frac{1}{16}$
L_2L_3	106 250	2 eye	1.03*	15 625	6.80	6.60	$6 \times 1\frac{3}{16}$
L_3L_3	120 450	2 eye	1.09*	15 625	7.70	7.06	$6 \times 1\frac{5}{61}$
U_1L_2	29 100	2 eye	0.60	12 500	2.33	3.88	$3\frac{1}{2} \times \frac{11}{16}$
U_2L_3	27 850	2 eye	0.59	12 500	2.23	3.80	$3 \times \frac{3}{4}$
U_3L_3	13 800	2 loop		12 500	1.10	1.05	$1\frac{1}{16} \times 1\frac{1}{16}$
U_3L_2	10 100	1 loop		12 500	0.81	1.00	$\frac{7}{8} \times \frac{7}{8}$
U_1L_1	17 950	2 eye	0.47	12 500	1.44	3.06	$3 \times \frac{1}{2}$

* $s = \frac{5}{4} \times 18\,000 = 22\,500$, and hence $t = 0.00316\, P^{\frac{1}{2}}$.

The counter U_3L_2 and one set of the main diagonals U_3L_3 are to be adjustable, turnbuckles being used, and the bars are to be upset according to § 105 of the Specifications.

ART. 108. VERTICAL POSTS.

POST U_2L_2. — From § 49 of the Specifications it is seen that the radius of gyration cannot be less than $(25 \times 12)/100 = 3$ inches, and the thickness of metal must be $\frac{5}{16}$ inch thick or more (§ 75, Cooper). The first condition precludes the possibility of using four angles latticed in pairs back to back, as their area would be greatly in excess of that required for a radius of gyration of 3 inches. A channel section is the most economical, although even in this case the area will be greatly in excess of that required. Remembering that the radius of gyration cannot be less than 3 inches and that the web cannot be less than $\frac{5}{16}$ inch thick, and noting that if the 3-inch legs of the floor beam connecting angles are used on the channel the width cannot be less than 8 inches, it is found that a 10-inch 20-pound channel is the section which can be used, and this satisfies all conditions.

The maximum stress in this post is live load only, as the dead-load stress is equal to zero, and its value is 20 800 pounds. The unit load allowable is, from the Specifications, $P = 10\,000 - 45\frac{l}{r}$. If the length of the post in feet be used, this formula becomes $P = 10\,000 - 540\frac{L}{r}$, where L is the length in feet. Here $L = 25$ feet, $r = 3.66$ inches, and the area $A = 2 \times 5.88 = 11.76$ square inches. Then $P = 10\,000 - 540\frac{25}{3.66} = 6300$ pounds per square inch, and $20\,800/6300 = 3.29$ square inches, are required. Thus it will be seen that, in order to meet the conditions of § 48 and § 75, the area is greatly in excess of that required by the given formula. A 6-inch 8-pound channel could be used if it were required to satisfy the conditions for unit load only.

POST U_8L_8. — For this post the total load is $17\,800 + (10\,000 + 4400/2)0.8 = 27\,560$ pounds (§ 51, Cooper). The channels used above will be tried. Here $L = 26.5$ feet, the unit load is

$P = 10\,000 - 540\dfrac{26.5}{3.66} = 6100$ pounds per square inch, and $27\,560/6100 = 4.52$ square inches are required. These channels will be used. An 8-inch 11.25-pound channel could be used, if the unit load formula alone were to be satisfied, and still would be in excess.

WIND ON VERTICAL POSTS.—The channels for these posts should be placed a certain distance from back to back, which will not only insure safety against the compression of the vertical load, but also that due to the effect of bending at the point where the transverse wind bracing is connected. This latter bending is caused by the wind. Fifteen feet of head room being required (§ 10, Cooper), and, considering the lower end of the post to be at the center line of the pin, the bending moment is $(150 \times 20 \times 15 \times 12)/4 = 135\,000$ pound-inches. This regards the post as fixed in direction at the ends and the upper lateral bracing as not in action, the point of contra-flexure being taken as half-way between the end of the post and the wind-bracing connection. It is the case of a member under compression and flexure. The following is an approximate method of determining the relation between the properties of the section and the loads which it may safely carry.

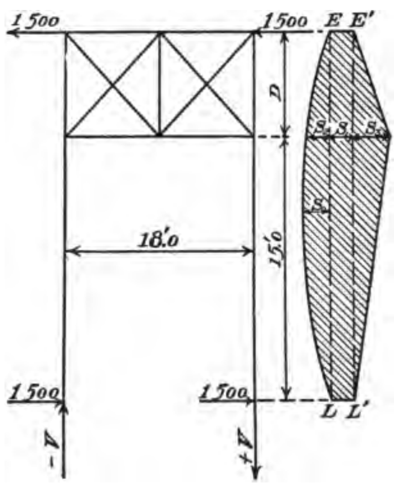


Fig. 138.

Let l be length of post, P the total load or stress it is to carry, A its sectional area, r the least radius of gyration of that section, P/A the direct unit stress S_1 due to P, and S the allowable compressive

unit stress for a short block. Let S_0 be the unit stress at the middle of the post, due to the bending that may be caused by P. Then $S = S_0 + P/A$ is the unit stress on the concave side at the middle. Now the straight line column formula is

$$S_1 = P/A = S - k\frac{l}{r},$$

in which $S = 10000$, $k = 45$ for the post in hand (§ 48, Cooper). Here the term kl/r is the value of S_0. Let S_2 be the flexural unit stress at any distance x from the end of the post. If the curve of bending moments or stresses be regarded as a parabola, this unit stress is

$$S_2 = \frac{4(l-x)x}{l^2}S_0 = \frac{4k(l-x)x}{lr}.$$

Now let M be the bending moment at the point x due to a wind force acting normal to the axis of the post. The unit stress caused by this force on the outer fiber of the post is

$$S_3 = \frac{Mc}{I},$$

in which I is the moment of inertia of the section and c is one-half its width normal to the channel webs. In order that the post shall be safe at the point considered it is necessary that $S_1 + S_2 + S_3$ shall not exceed the allowable unit stress S, or

$$\frac{P}{A} + \frac{4k(l-x)x}{lr} + \frac{Mc}{I} = S$$

is the equation to be satisfied by the properties of the section. Substituting for I its value Ar^2, and solving for c,

$$c = \frac{Ar^2}{M}\left(S - \frac{P}{A} - \frac{4k(l-x)x}{lr}\right), \qquad (1)$$

from which c can be computed for any assumed value of r.

Let b be one-half the distance from back to back of the channels, and g the distance from the back of a channel to an axis through the center of gravity of the channel section and parallel to its web. Let I_1 be the moment of inertia and r_1 be the radius of gyration of one channel section referred to this axis. Now $I = 2\ I_1 + Ah^2$, in which h is the distance from the axis of the post to the axis for which the moment of inertia is I_1. When the flanges of the channels are turned out, $h = b + g$, and when they are turned in, $h = b - g$. Substituting for h its value and for I_1 its value $\frac{1}{2}\ Ar_1^2$, there results

$$b = \pm g + \sqrt{r^2 - r_1^2},$$

in which the plus sign is to be used for flanges turned in, and the minus sign for flanges turned out. When flanges are turned in, as is to be the case with the posts of this bridge, the distance c is the same as b, and hence

$$r^2 = r_1^2 + (c - g)^2 \qquad (2)$$

is another relation between c and r.

The method of spacing the channels of the intermediate post is hence as follows: Assume a value of c, and compute r from (2); then insert this value of r in (1) and compute c. If the assumed and computed values of c do not agree, a new value is to be assumed and the computation repeated. Usually only two trials are necessary to bring the computed and assumed values within $2\frac{1}{2}$ per cent. The value of c to be assumed in the first computation should be a little greater than that value which gives the post section equal radii of gyration with respect to the two rectangular axes. If x and l are taken in feet, then $k = 12 \times 45 = 540$. For the case in hand (see Sheet 1, Fig. 137), $l = 26.5$ feet, $x = 11.5$ feet, $A = 11.76$ square inches, $P/A = 27\,560/11.76$ pounds per square inch, and $M = 135\,000 \times 2/3 = 90\,000$ pound-inches when reduced to live-load equivalent (see

additional clause, Art. 104). Assuming $c = 3.5$ inches, and taking $g = 0.61$ inch and $r_1 = 0.70$ inch from the handbooks for the given channel, there is found from (2) $r \doteq 2.98$ inches; inserting this in (1), there is then found $c = 3.39$ inches. Repeating the computation by assuming c as 3.55 inches, (2) gives $r = 3.02$ inches; inserting this in (1) gives 3.55 as assumed. The post channels will hence be placed $7\frac{1}{4}$ inches from back to back. If the channels should have their flanges turned out, the value of c would be 5.55 inches and the distance from back to back would be $5\frac{5}{8}$ inches.

The above is not an exact analysis, as the post is considered hinged at both ends when the unit load is computed and when the action of combined loads is considered, but in computing the bending moment due to wind it is considered fixed at one end. The assumption that the lateral bracing is not in action is also incorrect. The above, however, is a good approximate guide in aiding the designer to stiffen the post under wind.

ART. 109. HANGER AT THE HIP VERTICAL.

As the vertical L_1U_1 is a tension member, some provision must be made at L_1 to connect L_1U_1 to the floor beam. This is done by means of a hanger, which will consist of two side plates connected by a web and four angles. The function of this web is to transmit one-half the floor-beam reaction to the outer side, and on the foot-walk side to carry one-half the foot-walk reaction to the inner side. It must therefore resist a shear of $\frac{1}{2}(3000 + 5800 + 4200 + 34\,400) = 23\,700$ pounds. Its net thickness, assuming the depth the same as that of the floor beam, will be $23\,700/(20 \times 0.8 \times 10\,000) = 0.15$ inch, but by § 59 of the Specifications it must be at least $\frac{5}{16}$ inch thick, and this value will therefore be adopted.

The side plates should have a net area at the pin of 23 700/8000 or 2.96 square inches. As the pin is 4 inches in diameter,

the thickness of the plates, the width being taken as 10 inches, is $2.96/(10-4) = 0.493$ inch, and they will be made $\frac{1}{2}$ inch thick. The thickness of the angles must not be less than $\frac{5}{16}$ inch, the exact size to be determined by the conditions of detailing, but not less than $3 \times 2\frac{1}{2}$ inches.

ART. 110. END POSTS.

The section will consist of two channels, flanges turned out, a cover plate, and two flats. The flats will be riveted to the lower flanges, thus increasing the section, and at the same time keeping the neutral axis near the center of the web. The posts are spaced $7\frac{1}{4}$ inches from back to back, and, as all diagonals are packed inside the post, the top chord channels cannot be less than $7\frac{1}{4}$ inches from back to back. The distance from back to back will be made such that the channels of the posts will clear the rivet heads of the pin plates of the chord sections. A $\frac{7}{8}$-inch rivet head is $\frac{5}{8}$ inch high, and consequently $(7\frac{1}{4} + 2 \times \frac{5}{8})$ or $8\frac{1}{2}$ inches is the distance required. It is well to add at least $\frac{1}{4}$ inch for clearance, thus making the final distance from back to back of channels equal to $8\frac{3}{4}$ inches. The width of a 12-inch $20\frac{1}{2}$-pound channel flange is 3 inches, and using this, the cover plate width must be at least $14\frac{3}{4}$ inches. It will be taken $\frac{7}{16}$ inch thick and 15 inches wide, as plates over 6 inches should be ordered in variations of one inch in width. The flats will be taken as $3 \times \frac{1}{2}$ inches.

This section will now be investigated in order to determine if it fulfills the conditions, and does not give an excess or deficiency of area. The center of gravity is computed by taking moments about an axis through the center of top plate and parallel to its width. It is best to arrange the principal quantities in tabular form, A representing the area of any part in square inches, and l its lever arm in inches with respect to the axis mentioned above.

PIECE.	A	l	Al
2 channels	12.06	6.218	75.20
1 plate	6.56	0.0	0.0
2 flats	3.00	12.218	36.66
Sums	21.62		111.86

Then the distance from center of cover plate to the center of gravity of section is

$$g = \Sigma Al/\Sigma A = 5.21 \text{ inches},$$

and the eccentricity of the section, or distance from center of channel web to neutral axis is $e = (12/2 + \frac{7}{16}/2) - 5.21 = 1.008$ inches. The moment of inertia of the section is now computed, neglecting the moments of inertia of the plates about their own axes parallel to their width; thus

PIECE.	A	I'	h	Ah^2
2 channels	12.06	0	5.210	150
1 plate	6.56	256	1.008	12
2 flats	3.00	0	7.008	146
Sums	21.62	256		308

whence $I = \Sigma(I' + Ah^2) = 564$ inches^4, and the radius of gyration of the section is

$$r = (564/21.62)^{\frac{1}{2}} = 5.1 \text{ inches}.$$

Lastly, by the column formula of the Specifications,

$$P = 10\,000 - 540\frac{28.28}{5.1} = 7060 \text{ pounds per square inch},$$

which is the safe unit load for the assumed section. As the stress on the post is 152 100 pounds, the area required is 152 100/7060 = 21.54 square inches, which is practically the same as that assumed, and accordingly the latter may be used.

By § 97 of the Specifications, it will be seen that the thickness of the channel webs is less than $\frac{5}{16}$ inch. The discrepancy being small, however, their use will be allowed, as much economy in quantity of material results.

The end post is also subjected to bending, due to wind, at a point where the knee brace of the portal strut joins it. If the portal strut be taken as six feet deep and the knee brace as joining the post six feet lower, the bending moment is

$$12\ [(28.3 - 12) \times 3 \times 20 \times 150]/2 = 146\,700 \text{ pound-inches},$$

when the post is free at lower end, or $146\,700/2 = 73\,350$ pound-inches if the post is fixed at the lower end, the point of contraflexure being considered as half-way between the end and the knee-brace connection.

The end post may be regarded as fixed if the moment of the wind acting with a lever arm equal to the distance from center to center of end pins is less than the moment caused by one-half the stress in the end post acting with a lever arm equal to the distance from center to center of the bearings of the pin at the lower end. For this computation the length of the end post may be considered as 28.3 feet and the distance from center to center of bearings as slightly more than $8\frac{3}{4}$ inches, say $9\frac{1}{2}$ inches. The moment for the first case is $3000 \times 3 \times 28.3 \times 12 = 3\,060\,000$ pound-inches, and the moment for the second case is $\frac{1}{2}(9\frac{1}{2} \times 152\,700) = 725\,000$ pound-inches. As the first of these values is greater than the second, the post will be considered as having free ends and will be required to stand a bending moment of $146\,700$ pound-inches or $\frac{2}{3}\,146\,700 = 98\,000$ pound-inches when reduced to live-load equivalent (Art. 104, additional clause to § 48, Cooper).

The moment of inertia of the section with reference to an axis perpendicular to the cover plate is now computed and is found to be 547 inches^4, thus giving a radius of gyration of 5.02

inches, and an allowable unit load of 6960 pounds per square inch. Hence, 152 100/6960 = 21.8 square inches are required, but the assumed section will not be changed, as it is less than one percent in deficiency. Referred to the above axis the section can stand a live-load moment of

$$M = \frac{SI}{c} = \frac{10\,000 \times 547}{7.5} = 731\,000 \text{ pound-inches.}$$

Since the moment due to the wind, 98 000 pound-inches, is less than one-fourth of 731 000, it need not be considered (§ 52, Cooper).

ART. 111. TOP CHORD SECTIONS.

For the chord U_1U_2 a 5/16 × 15 inch cover plate and two 12-inch 20½-pound channels will be used (§ 90, Cooper). Here, proceeding as in the case of the end post, $g = 4.44$ inches, $e = 1.72$ inches, $I = 384.5$ inches^4, $r = 4.79$ inches, and the total area is 16.75 square inches. Then (§ 48, Cooper),

$$P = 12\,000 - \frac{660 \times 20.6}{4.79} = 9140 \text{ pounds per square inch,}$$

and 147 750/9140 = 16.20 square inches is the area of section required. The moment of inertia referred to an axis through the middle of the section and perpendicular to the cover plate is 406.9 inches^4, and hence the assumed section is amply safe in that direction. The wind stresses are not considered in any of the top chord sections (§ 52, Cooper).

The same section will be used for both U_3U_2 and U_3U_3, and will be designed for the greatest stress, which is 163 350 pounds. A ⅜-inch cover plate and two 12-inch 20-pound channels will be tried. Here, computing as before, $g = 4.22$, $e = 1.965$, $I = 404$ inches^4, $r = 4.77$ inches, and the total area is 17.69 square inches. The unit load allowable is

$$P = 12\,000 - \frac{660 \times 20}{4.77} = 9240 \text{ pounds per square inch,}$$

and $163\,350/9240 = 17.7$ square inches is the area required. The moment of inertia referred to an axis through the center of the section and perpendicular to the cover plate is 444.5 inches^4, which shows the section to be safe for that axis, and hence it will be used.

ART. 112. CENTER LINE OF PINS.

Pins are not placed at the centers of gravity of the sections, nor on the center line of the web of channels. They are placed at such a distance below the center of gravity that the direct stress acting along the neutral axis will produce a moment neutralizing the moment due to the weight of the member itself. Let this distance be denoted by p, let W be the total weight of the member in pounds, l the length in inches, and P the total stress in the member, which in this case is the sum of the dead and live load stresses. Then

$$Pp = \tfrac{1}{8}\,Wl, \text{ or } p = \tfrac{1}{8}\,Wl/P.$$

Let d be the distance of center line of pins above the center of web of channels. Then

$$d = e - p.$$

To determine the weight per linear foot of a member for this computation, the weight of material in the section is taken and 20 percent added for the weight of batten plates, lattice bars, rivet heads, and pin plates. For example, for the end post L_0U_1 the weight per linear foot is,

2 channels, 12 inch $\times$ $20\frac{1}{2}$ pounds =	41 pounds,
1 plate, 15 $\times$ $\frac{7}{16}$ inches =	22 pounds,
2 flats, 3 $\times$ $\frac{1}{2}$ inches =	10 pounds,

and the sum of these plus 20 percent is 100 pounds nearly. Here the component which causes bending is 100/1.414 or 71

pounds, and the total weight W is 71×28.3 pounds. The distance p is

$$p = \frac{71 \times 28.3 \times 28.3 \times 12}{8 \times 179\,700} = 0.475 \text{ inch},$$

and hence $d = 1.008 - 0.475 = 0.523$ inch is the correct distance of the center line of pins above the center line of web of channels. In like manner are found $p = 0.25$ inch, and $d = 1.47$ inches for U_1U_2, while for U_2U_3 and U_3U_3 there results $p = 0.23$ inch, and $d = 1.74$ inches. As loads increase or decrease, d increases or decreases. The center line of pins must also be the same distance from the center of the web throughout for constructive reasons. It is not advisable to use the highest or the lowest values of d, but an average value, say $1\frac{1}{4}$ inches, should be taken.

ART. 113. DESIGN OF PINS.

The pin at each joint should be designed to resist bending, bearing, and shear, and also to satisfy § 104 of the Specifications. As an example a pin will be designed for the point L_3 of the lower chord. Here the large eye-bars being the members carrying the largest stress, the greatest bending moment will occur when they take the maximum stress, which will be when the bridge is entirely loaded. The maximum stress will be taken as the live-load equivalent (§ 48, Cooper). For this loading the stress in L_3U_3 is zero; the stress in U_2L_3 is $+ 11\,000$ from dead and $+ 24\,900$ from live load; the stress in U_3L_3 is $+ 4400$ from dead and $+ 10\,000$ from live load; and the floor beam exerts a downward pull of 1300 pounds from dead and 29 400 pounds from live load. The wind load is taken as 48 000 pounds in each member, in order to balance the horizontal components.

A table should now be prepared giving the horizontal and vertical components of these stresses for the point L_3. It is to

	Horizontal Components, Pounds.			Vertical Components, Pounds.		
	L_2L_3	L_3L_3	U_2L_3	U_3U_3	U_2L_3	Lower end of U_3L_3
Live	−117 500	+133 100	−15 550	+10 000	+19 400	−29 400
½ dead	−26 000	+29 400	−3 450	+2 200	+4 300	−6 500
⅔ wind	+32 000	+32 000	− 000	+ 000	+ 000	− 000
Sum	−175 500	+194 500	−19 000	+12 200	+23 700	−35 900

be noted that the sum of the horizontal components and the sum of the vertical components are each equal to zero. This serves as a check on the computations.

Taking the packing from the stress sheet (Fig. 137) and assuming the total thickness of the bearing surface of U_3L_3 to be $\frac{1}{2}$ inch, and cutting the flanges of the channels to within $1\frac{1}{4}$ inches of the backs, the horizontal bending moment at the center of each bar is computed by $M = M + V'x$ (Mechanics of Materials, Art. 47). Thus the horizontal bending moments are found as follows:

Member.	Stress.	V'	x	$V'x$	M
L_2L_3	−87 750				
		−87 750	1.313	−115 000	
L_3L_3	+97 250				−115 000
		+ 9 500	2.345	+ 22 000	
U_2L_3	− 9 500				− 92 800
		± 000	——		

while the vertical bending moments are:

Member.	Stress.	V'	x	$V'x$	M
U_3L_3	+11 850				
		+1850	1.438	+17 100	
U_2L_3	−11 850				+17 100
		± 000	——	± 000	

The resultant bending moment under U_2L_8 is

$$(92\,800^2 + 17\,100^2)^{\frac{1}{2}} = 93\,600 \text{ pound-inches.}$$

As this is less than 115 000, the bending moment under L_8L_8, the maximum is therefore 115 000 pound-inches, which occurs under the large eye-bar L_3L_8. The size of the pin can now be computed according to methods given in text-books on mechanics of materials, or by reference to tables in manufacturers' hand-books, and it will be found that a 4-inch pin is needed to resist the bending moment. By § 104 of the Specifications the pin is required to be $6 \times \frac{3}{4} = 4\frac{1}{2}$ inches in diameter. The maximum unit shear is $87\,750/15.9 = 5500$ pounds per square inch. The unit stress per square inch for bearing for L_2L_8 is $87\,750/(4.5 \times 1\frac{3}{16}) = 16\,400$ pounds; in the same manner that for L_3L_8 is 16 350 pounds, and that for U_2L_8 is 8200 pounds, all of which show the pin to be safe. Hence the $4\frac{1}{2}$-inch pin will be used at L_8 and L_2.

Upon computing pins for other joints it is found that a $3\frac{3}{4}$-inch pin can be used at L_1, that a 4-inch pin is required at L_0, a $3\frac{1}{4}$-inch pin at U_1, a $3\frac{1}{2}$-inch pin at U_2, and a 3-inch pin at U_8. A $4\frac{1}{2}$-inch pin will be used at L_8 and L_2, a 4-inch pin at L_0 and L_1, and a $3\frac{1}{2}$-inch pin at U_1, U_2, and U_8.

As the center line of pins is $1\frac{1}{4}$ inches above the center line of the webs of the channels, and as the eye-bars should have a section through the center of pins of 40 percent excess over the body of the bar, the head of a 5-inch eye-bar is $5 + 4 + (0.4 \times 5) = 11$ inches wide. The radius of the head is therefore $5\frac{1}{2}$ inches, which shows that the head of the bar will strike the cover plate. If the cover plate is stopped off a few inches from the end, this obstruction will be cleared, and in doing so the strength of the member will in nowise be lessened, as at the ends the allowable unit load is the allowable unit stress in bearing, or 18 000 pounds per square inch, and the 15.06 square inches left in the

post is capable of carrying $18\,000 \times 15.06 = 270\,100$ pounds, the pin plates not being considered. The cover plate will be so arranged in detailing.

ART. 114. PEDESTALS AND ROLLER NESTS.

The vertical plates as seen on the stress sheet will go inside of the end post. The maximum reaction is equal to $3\frac{1}{2}$ times the dead panel load divided by 2, plus 3 times the live panel load. The half panel live load that comes at L_0 is transferred directly to the abutments by the end stringers, there being no floor beam at the end. The maximum reaction is, therefore,

$$\tfrac{1}{2}(3\tfrac{1}{2} \times 13\,000) + 3 \times 29\,400 = 115\,950 \text{ pounds.}$$

The design of the pedestals for the fixed end will be made first. The bearing area required is $115\,950/18\,000 = 6.45$ square inches, and $\frac{1}{4}(6.45) = 1.612$ inches is the width of the bearing area on a 4-inch pin. Two vertical bearing plates each $\frac{7}{8}$ inch thick will be used. The inside connection angles will be $5 \times 3 \times \frac{1}{2}$ inches, the 5-inch leg vertical, and the outer ones will be $5 \times 3\frac{1}{2} \times \frac{1}{2}$ inch (§§ 130–132, Cooper). The masonry plates cannot be less than $8\frac{3}{4} + 2 \times 3\frac{1}{2} = 15\frac{3}{4}$ inches in width, say 16 inches. The bearing area required is $115\,950/250 = 465$ square inches.

The length of the masonry plate must be $465/16 = 29$ inches. The bearing plate will be the same area and thickness. Both bearing and masonry plates will be ordered $\frac{13}{16}$ inch thick and finished on one side to $\frac{3}{4}$ inch. The pedestal will be anchored to the masonry by $1\frac{1}{4}$-inch anchor bolts securely fox-bolted in the masonry to a depth of 12 inches.

The design of the pedestal and roller nest for the free end is as follows: The vertical plates and connection angles will be the same as at the fixed end. The width cannot be less than $15\frac{3}{4}$ inches, as before, nor can it be greater than $21\frac{3}{4}$ inches,

since the bearing plate should not extend, unsupported, beyond the edges of the connecting angles for a distance greater than 3 inches. If it extends further than 3 inches, the remainder cannot be considered in taking up the bearing. The allowable load for rollers per linear inch is 300 d (§ 27, Cooper). Here d is $2\frac{7}{8} + 0.4 \times 1 = 2\frac{7}{8} + \frac{3}{8} = 3\frac{1}{4}$ inches, which makes the load per linear inch equal to 975 pounds. Hence, 115 950/975 = 119 linear inches are required. If each roller be 15 inches long, eight will be needed. Allowing for a small guide bar 2 inches wide at the middle, the rollers will be 17 inches long. If a tie bar $\frac{1}{2}$ inch thick be used on each side and guide angles 3×3 inches, the masonry plate, allowing $\frac{1}{16}$ inch clearance between members, will be at least $(17 + 4 \times \frac{1}{16} + 2 \times \frac{1}{2} + 2 \times 3) = 24\frac{1}{4}$ inches wide, and it will be taken as 25 inches. If a $\frac{1}{8}$-inch space be allowed between each roller, and $\frac{3}{4}$-inch tie rods be used, and a variation of 150° in temperature be assumed, the length of the plate will be found to be $31\frac{1}{4}$ inches. The bearing and masonry plates should be ordered $\frac{13}{16}$ inches thick and finished on one face to $\frac{3}{4}$ inch. The dimensions of the bearing plate should be the same as the masonry plate, the extra width being required in order that room may be provided to allow slotted holes to be cut for the anchor bolts. In detailing, care should be exercised to extend the bearing plates properly and to make the under side the same distance below the center line of the pins as the bottom of floor beam, plus a $\frac{5}{16}$-inch connection plate, in order to allow for the connection of the angles of the lower lateral bracing.

ART. 115. LATERAL AND TRANSVERSE BRACING.

According to § 4 of the Specifications all laterals must be of shapes capable of resisting compression. Angles will be used, but for a tension member the section will be determined from the tensile stresses as computed.

LOWER LATERALS. — Here 18 000 pounds per square inch is the allowable unit stress, and dividing this into the computed stresses, it is found that sectional areas of 1.51, 1.04, 0.62, and 0.22 square inches are needed in the first, second, third, and fourth panels respectively. The angles must, however, have a net area of not less than $\frac{3}{4}$ of a square inch (§ 97, Cooper). A $3\frac{1}{2} \times 3 \times \frac{5}{16}$ inch angle gives a net area of 1.55 square inches, after allowing for one $\frac{7}{8}$-inch rivet taken out of the section. The $3\frac{1}{2}$-inch leg will be placed vertically downward. It may be here stated that the vertical leg of an angle should always be placed downward when possible, for the water will run off quicker, and dust and dirt do not accumulate and hasten deterioration as they do when the leg is upwards, while a trough carrying the rain and dirt into the connection at the ends of angle is also avoided.

UPPER LATERALS. — The computed stresses being small, in all cases requiring less than $\frac{3}{4}$ of a square inch, $3 \times 2\frac{1}{2} \times \frac{5}{16}$ inch angles will be used and connected to the top chord by $\frac{5}{16}$-inch plates and $\frac{3}{4}$-inch rivets.

INTERMEDIATE TRANSVERSE BRACING. — By § 121 of the Specifications transverse or sway bracing is required. The computed stress for its lower chord is $(3000 \times 26.5)/(2 \times 11.5) = 3500$ pounds. The radius of gyration cannot be less than $(12 \times 18)/120 = 1.8$ inches. Two $4 \times 3 \times \frac{5}{16}$ inch angles, placed $\frac{1}{2}$ inch from back to back, give a radius of gyration of 1.9 inches and conform to the Specifications in regard to thickness. They will be used, although they give an excess of about one-half a square inch of area over that required. The top chord of the intermediate bracing will consist of four of these angles latticed, and the bottom chord will consist of two. The longer leg is placed outward in each case. These two chords will be connected by two panels of latticing consisting of $3 \times 3 \times \frac{5}{16}$ inch angles. These angles, designed to resist the vertical shear,

require a much smaller section, but by § 97 of the Specifications they must be used.

ART. 116. PORTAL BRACING.

In order to give the required head room, the portal bracing can be 7 feet deep. It will, however, be taken as 6 feet, with a knee brace joining the post 6 feet farther down. The wind load at the hip is $150 \times 20 \times 3 = 9000$ pounds, and the vertical shear is $\frac{1}{18}[3000(26.5 + 25 + 20)1.414] = 16\,850$ pounds. The moment in the portal strut at the point where the knee brace joins it, is $9000 \times 72 + 4500 \times 267.6 - 16\,850 \times 66 = 738\,000$ pound-inches, and its stress, taking 6 feet as effective depth, is $738\,000/72 = 10\,200$ pounds. By § 48, the radius of gyration cannot be less than 1.8 inches and, by § 97, the thickness of the angles cannot be less than $\frac{5}{16}$ inch. For the bottom chord, the stress, where the knee brace joins it, is $(4500 \times 28.3 \times 12)/72 = 16\,600$ pounds. Two angles $3\frac{1}{2} \times 3 \times \frac{3}{4}$ inches, spaced $\frac{3}{4}$ inch from back to back, will be tried. Here $r = 1.90$ inches, and the net area $= 7.12$ square inches. The unit load is $13\,000 - 720\,l/r$, l being in feet. This gives 6150 pounds per square inch for the unit load, and dividing this into the total stress gives 2.7 square inches as the area required, which shows that the angles are too heavy. Two $3\frac{1}{2} \times 3 \times \frac{5}{16}$ inch angles have a radius of gyration of 1.8 inches. Here $P = 5800$ pounds per square inch and 2.86 square inches area required. These angles give a gross area of 3.86 square inches and a net area of 3.22 square inches, and they will be used. Both flanges will be made of the same section.

The knee brace of the portal strut will now be investigated. Considering that it is to be placed at an angle of 45 degrees, the length is $1.414 \times 6 = 8.5$ feet. The stress in it is $(4500 \times 22.3)/4.25 = 23\,700$ pounds, and two $5 \times 3 \times \frac{5}{16}$ inch angles give a least radius of gyration of 0.85 inch. Hence, $P = 5800$ pounds

per square inch and the net area required is 4.10 square inches. As these angles give a gross area of 4.80 and a net area of 4.18 square inches, they will be used.

ART. 117. THE STRESS SHEET.

The work of the designer is now finished, and the results, with sketches representing the general arrangement of stringers, floor beams, eye-bar packing, and details, are handed to a draughtsman in his office, who makes the stress sheet (Fig. 137), often improperly called a strain sheet.

On this sheet the stresses in the members, together with their sections, are noted on outline diagrams of the truss and lateral systems drawn to a small scale. Another view, one-half of which shows an end elevation of the truss and the other half a section taken near the middle of the span, is drawn to a larger scale. On this view are shown as many of the details and arrangements of the floor system and truss members as possible. A diagram of the packing of the eye-bars is also given. The eye-bars of L_1U_1 are placed outside of the chord at the hip U_1 in order to reduce the bending stress on the pin. For the same reason the largest eye-bars at the middle of the span are packed nearest to the posts, and the diagonals inside of the posts.

The roadway is arranged according to §§ 17–21 of the Specifications. The sidewalk has been placed below the level of the roadway in order to lessen the tendency of persons to step from the sidewalk to the roadway while crossing. Bridge companies do not have the facilities for the manufacture of ornamental railings, but these are usually bought from firms doing that class of work. These firms furnish the bridge companies with sketches, showing the location, size, and number of holes for the bolts or rivets which connect the railing to the bridge, from which the details of the outer foot-walk stringers and the bracket may be correctly made.

The distance from base of rail to the masonry and from base of rail to the center line of pins, together with clearances and general dimensions of the truss and floor system, are also placed on the stress sheet.

This stress sheet may also be used as a marking diagram, using the following notation. The stringers are marked with the letters S_1, S_2, S_3, etc., and if one differs from the other it should be marked S_1' or S_1x. Should two be similar in all respects except that one is right-handed and the other left-handed, they are marked S_1R and S_1L. Top laterals should be marked T_1, T_2, etc., bottom laterals L_1, L_2, etc., transverse bracing TV_1, TV_2, etc., and plates P_1, P_2, etc., the same rules in regard to subscripts, primes, and rights and lefts applying here also. It is best to give the moments and shears for the floor system in order to save their re-computation by the draughtsman who details the bridge. A good clear stress sheet is indispensable to enable the details to be made correctly and quickly.

This sheet, together with a copy of the Specifications, is now sent to the detailing room, where it is placed in the hands of a draughtsman, who makes the details according to these specifications and the practice of the bridge company.

Art. 118. Detailing the Bridge.

The full set of drawings for this highway bridge comprises ten sheets, of which only five are here published. The size of the original drawings between border lines was 15 × 25 inches, the lettering being made somewhat larger than usual in order to permit of satisfactory reduction. These drawings are as follows:

Sheet 1. Stresses and Sections (Fig. 137).

Sheet 2. Stringers and Floor Beams (Fig. 139).

Sheet 3. Intermediate Posts (Fig. 140).

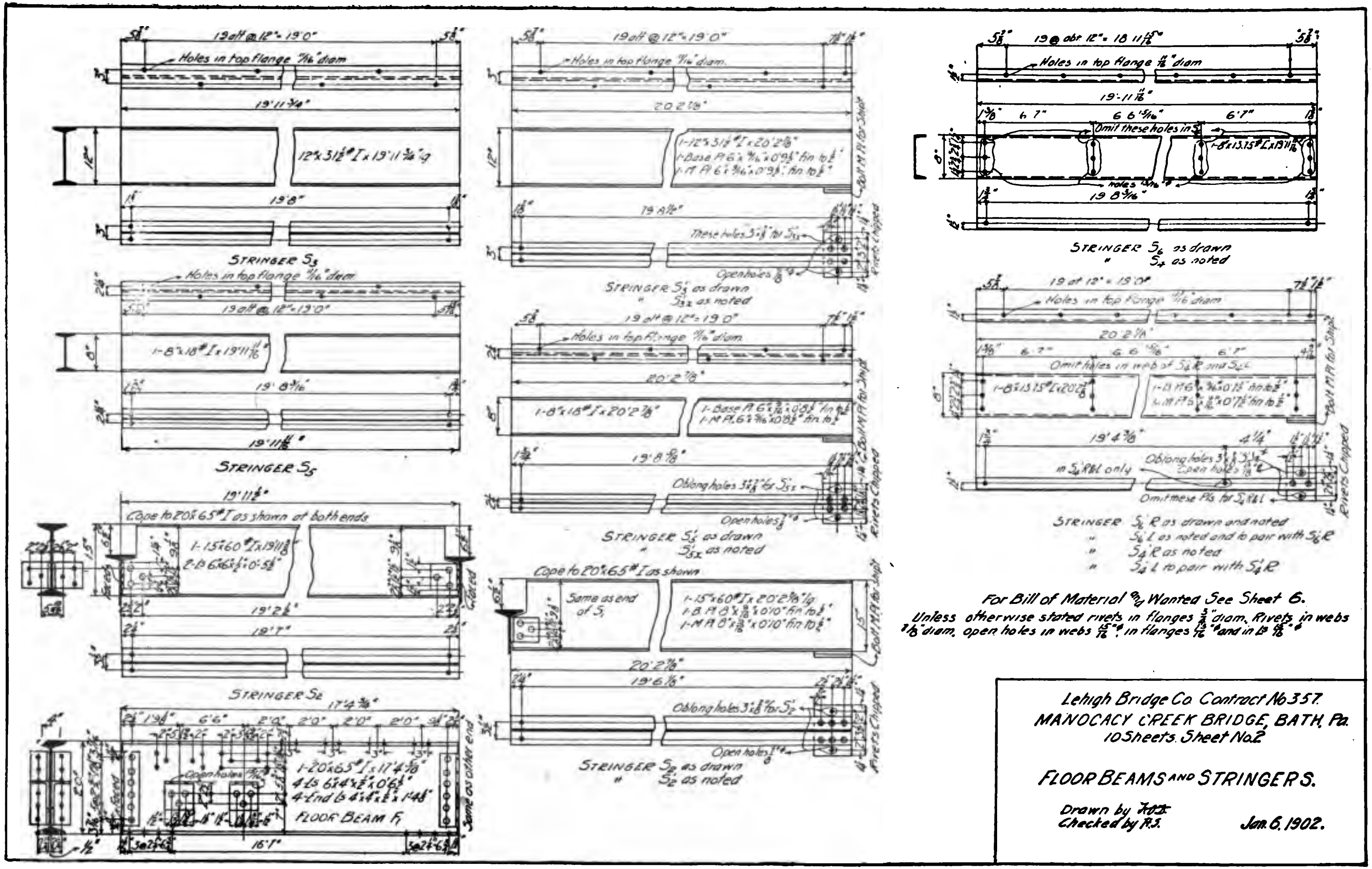

STRINGER S3
STRINGER S5
STRINGER S2
FLOOR BEAM F1
Cope to 20" 65# I as shown at both ends
STRINGER S'3 as drawn
" S'3x as noted
STRINGER S'5 as drawn
" S'5x as noted
STRINGER S'2 as drawn
" S'2 as noted
STRINGER S6 as drawn
" S4 as noted
Omit these holes in S4
Holes in top flange 7/16" diam
Rivets Chipped
Bolt M.R. for Shipt
For Bill of Material Wanted See Sheet 6.
Unless otherwise stated rivets in flanges 3/4" diam. Rivets in webs 7/8" diam. open holes in webs 15/16", in flanges 13/16" and in Ls 15/16"
Lehigh Bridge Co. Contract No 357.
MANOCACY CREEK BRIDGE, BATH, Pa.
10 Sheets. Sheet No 2
FLOOR BEAMS AND STRINGERS.
Drawn by
Checked by R.S.
Jan 6. 1902.

Fig. 139.

Sheet 4. End Posts and Top Chords U_8U_8 (Fig. 141).

Sheet 5. Top Chords U_1U_2 and U_2U_8.

Sheet 6. Portal Bracing and Foot-walk Bracket.

Sheet 7. Pedestals and Roller Nests (Fig. 142).

Sheets 8, 9, and 10. Top Laterals, Bottom Laterals, and Transverse Bracing.

In addition to these drawings bills for eye-bars, loop and adjustable eye-bars, field rivets, and bolts are prepared on printed forms, on which are noted the final dimensions and lengths, and also the additional lengths needed to make the heads of bars. If the bar is an adjustable member, the additional length for the upset for the screw is given and a note is made stating whether a turnbuckle or a sleeve nut is to be used. The field-rivet bill gives the number required of each diameter and also the length of shank required for the necessary grip.

On each sheet of detail drawings there is usually placed a Bill of Material and a "Wanted" list, both in tabular form, but such tables are omitted on Figs. 140–142 on account of lack of space. When a sheet is crowded a note may be made, as seen on Fig. 139, that these lists are given on one of the following sheets.

SHEET 2. FLOOR BEAMS AND STRINGERS. — Unless otherwise stated, the rivets used in the flanges are $\frac{3}{4}$ inch and those in the webs $\frac{7}{8}$ inch in diameter. The beams are drawn to scale in depth and width, but are shortened in the direction of length in order to place a large number on one sheet. For each beam there is shown the top view, side elevation, and transverse and longitudinal sections. The holes in the top flanges are for $\frac{5}{8}$-inch bolts, with which the $1\frac{1}{2}$-inch nailing strips are connected. The holes at the ends of the lower flanges are for rivets which connect the stringers to the floor beams. A clearance of $\frac{1}{4}$ inch is allowed between the ends of roadway stringers, and

$\frac{5}{16}$ inch is allowed between the ends of foot-walk stringers. All end stringers, except that of the inner foot-walk, have masonry and bearing plates. For the fixed end of the bridge open holes $\frac{3}{4}$ inch in diameter are left in these plates for the anchor bolts; for the roller end slotted holes are cut to allow for movement due to temperature. The rivets shown, which of course only go through the bearing plate, are countersunk and chipped on the side next to the masonry plate. The end inner foot-walk stringer is seen connected to the end post by a bent plate. This arrangement is necessary, as the shoe of the truss prevents the masonry of the pier from being built close enough to allow the end of the stringer to bear upon it. The distance of the rivet hole from the end of the stringer can either be computed or the detail laid out to a large scale and measured off directly. The masonry plate is usually shipped bolted to the bearing plate as noted. The size of the plates will, in most cases, exceed those previously calculated.

The trolley stringers, being of greater depth than those of the roadway, are connected in the manner shown in order to bring the tops to the same elevation. The top flange and part of the web is cut to allow the flange of floor beam to fit in; this operation is called coping. The reaction of the trolley stringer is 20 370 pounds, and there are six field and three $\frac{7}{8}$-inch shop rivets in single shear. Their aggregate strength is $3200 \times 6 + 4800 \times 3 = 33\,600$ pounds, which shows the joint to be amply strong in shear. The strength of the joint in bearing is that of the three shop rivets in web of stringer and the three in web of floor beam, and is $3 \times 6280 + 3 \times 0.59 \times 0.875 \times 14\,400 = 43\,840$ pounds, showing it to be safe in bearing. The thickness of the angles must be sufficient to take the bearing stress. It is not the best practice to connect a leg of an angle with only two rivets, hence three are used, although giving an

excess of strength. The small shelf angle should always be used, if possible, even if not required for strength, as it is of great convenience in erection, enabling the field rivets to be driven without blocking up the stringer. It is to be noted that where connection angles are used the beam is cut $\frac{1}{16}$ inch short at each end and the angles faced off true to length (§ 116, Cooper). The end reaction of the floor beam is 34 400 pounds. The number of shop rivets required in bearing in the $\frac{1}{2}$-inch web of the floor beam is 34 400/6280 = 6; the number of field rivets required in the end connection in single shear is 34 400/3200 = 11; the number of field rivets required in bearing in the 0.382-inch web of the post channels is 34 400/3200 = 11, but 12 will be used. In the bottom flange of the floor beam are holes to receive the plate of the lower lateral connection; this connection should be plotted to scale to determine the size of the plate. The component of stress in the end panel tension member parallel to the floor beam is 18 × 27 180/26.9 = 18 200 pounds, and as a $\frac{3}{8}$-inch plate is used, the joint will be weak in shear and 18 200/2360 × 1.4 = 6 is the number of $\frac{3}{4}$-inch rivets required in single shear. On account of the large size of the plate 8 rivets are used in order that the unsupported width shall not be too great.

SHEET 3. INTERMEDIATE POSTS AND HANGERS. — As the posts U_2L_2 and U_3L_3 only differ in length, they are detailed together. The details, such as batten plates and lattices, are the same, and thus only two sets of dimensions are needed, one drawing doing for both. The batten plates and lattices are determined by § 111 of the Specifications. The batten plates should be so placed that they do not interfere with the diagonals. The maximum stress which can come on the pin at the lower end is 20 000 + 29 400 + 1300/2 = 56 780 pounds. By § 54 of the Specifications 56 780/(18 000 × 4.5) = 0.692 inch is the width of bearing area required on a 4.5-inch pin. As the thickness of the channel web

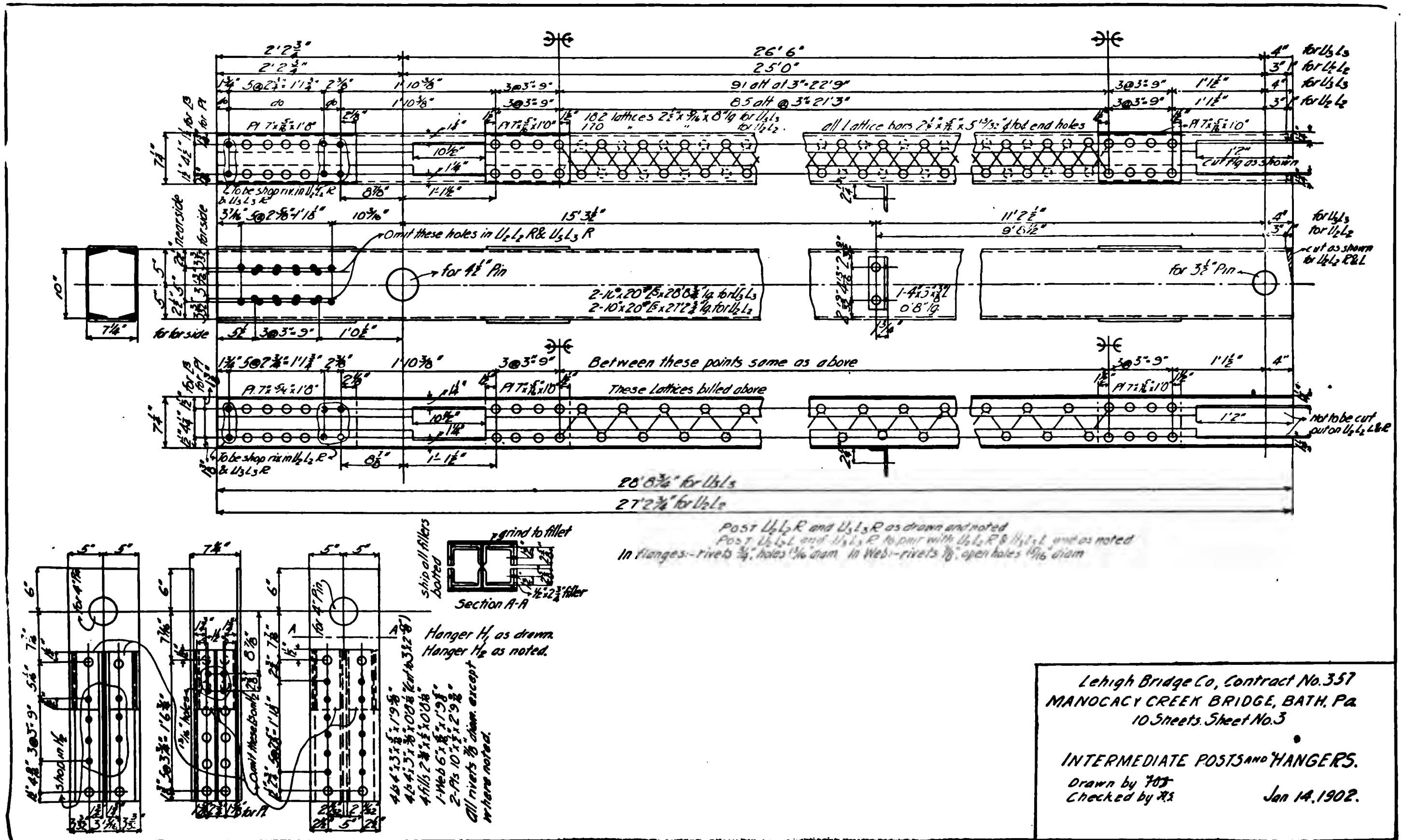

Lehigh Bridge Co, Contract No. 357
MANOCACY CREEK BRIDGE, BATH, Pa.
10 Sheets. Sheet No. 3
INTERMEDIATE POSTS AND HANGERS.
Drawn by
Checked by
Jan 14, 1902.
Omit these holes in U_2L_2 R & U_3L_3 R
for 4½" Pin
for 3½" Pin
Between these points same as above
These Lattices billed above
all Lattice bars 2½" x ⁵⁄₁₆" x 5' ¹³⁄₃₂" c/tos end holes
Post U_2L_2 R and U_3L_3 R as drawn and noted
In flanges:–rivets ¾", holes 1⅛" diam. In Web:–rivets ⅞", open holes ¹⁵⁄₁₆" diam.
Section A-A
grind to fillet
ship all fillers bolted
Hanger H_1 as drawn.
Hanger H_2 as noted.
All rivets ⅞" diam. except where noted.

Fig. 140.

is 0.764 inch, no pin plates are required. The maximum stress on the $3\frac{1}{2}$-inch pin at the top is 20 800 pounds, and by a similar computation 0.32 inch is the total thickness required, showing that no pin plates are required. The flanges of the channels are cut to allow the diagonals to be placed closer to the channel web, and thus reduce the bending on the pins. This of course weakens the section, and an investigation as to strength should be made. The maximum tensile strength which U_8L_8 must be designed to resist (§ 51, Cooper) is $(10\,400 + 4400/2) \times 1.8 =$ 22 000 pounds. By cutting off the flanges the area is reduced by 2.2 square inches, and this is further reduced by 2.68 square inches on taking out the section of the $3\frac{1}{2}$-inch pin. This leaves 6.88 square inches, which is capable of standing $6.88 \times 12\,500 = 86\,000$ pounds, showing the section to be safe.

At the lower end two $\frac{5}{16}$-inch plates are riveted to the post, one on each side. These take the place of a diaphragm, and their function is to cause both channels of the post to take an equal amount of load. The total number of rivets required, on the foot-walk side, in one side of both plates, must be sufficient to transfer one-half the vertical stress in the post from one side to the other. This shear is equal to one-half the sum of the maximum shears of the foot-walk and floor beam, or $\frac{1}{2}(13\,000 + 34\,400) = 23\,700$ pounds. This requires seven $\frac{3}{4}$-inch shop rivets, or 4 on each side of each side plate, but more are used to keep the rivet spacing less than 5 inches. These plates should be as long as the depth of the floor beam, and the top should be even with the top of floor beam, and clearing the heads of the eye-bars of lower chord. In these plates are holes for the connection of floor-beam and foot-walk bracket; the posts opposite the foot-walk side of bridge have these foot-walk holes omitted. The number of rivets for the floor-beam connection was computed in Art. 106. The number required for the shear of the foot-walk bracket is six $\frac{3}{4}$-inch field rivets in single bearing in

the $\frac{5}{16}$-inch plate. The open holes in these plates, in the flanges of channels, and in the posts on the foot-walk side of bridge, are for the connection of foot-walk bracket, which consists of 3×3 inch angles at bottom and 6×3 inch angles at top, riveted to the flanges of channels, in order to have rivets in shear only, and not in tension, as is too often the case. The small angle on the inner side of the post is for the connection of the transverse bracing, and should have holes to take the standard gage of the bottom flange angles of the bracing. It should also be placed at a distance of 15 feet above base of rail (§ 10, Cooper).

The hanger shown is simply a counterpart of the lower end of the post, except the side plates. Its function is the same as the end of post, the diaphragm web transmitting one-half the shear. Here $\frac{7}{8}$-inch instead of $\frac{3}{4}$-inch rivets are used. The required thickness of web is $23\,700/(8000 \times 20) = 0.148$ inch (§ 53, Cooper). A $\frac{5}{16}$-inch web must be used (§ 97, Cooper). The number of rivets required in bearing in this web, to connect the angles to the diaphragm web, is $23\,700/3930 = 6$. The number of rivets required to connect angles to side plates is $23\,700/3930 = 6$. More than the required number are used on each side, in order to allow for the floor-beam and foot-walk bracket connections, and in order to keep the rivet spacing within the allowable limit. The connections of the floor beam and foot-walk should be so arranged that their tops may be the same distance below the center line of pins as those which are connected to the posts. To allow for the foot-walk connection, angles are riveted on and brought out level to the side plates by means of small fillers; these will of course be omitted on the other side of the bridge. The number of rivets required in the flange connection of the foot-walk is determined as follows. The maximum top-flange stress in the foot-walk bracket is 23 700 pounds, and the bearing value of a $\frac{3}{4}$-inch field rivet in a $\frac{5}{16}$-inch plate at a bearing value of 12 000 pounds per square inch is 2810 pounds.

The number of rivets required is 23 700/2810 = 8.5, and 8 will be used. At the bottom four rivets are used to keep the bracket from lateral motion.

The box form of post is expensive from a shop point of view on account of the difficulty in riveting, but it is better in appearance, and has no outstanding edges to interfere with traffic. It also has the advantage over others of throwing the material in the web nearer to the outer surface of post, and thus allowing a closer compliance with § 102 of the Specifications.

SHEET 4. END POSTS AND TOP CHORD U_8U_8.—The end post will first be considered. According to § 48 and § 133, the end post is not a top chord, and hence camber will not be considered. The batten plates are required to be $1\frac{1}{2}(12+\frac{1}{2}+\frac{7}{16})$ $= 19\frac{3}{4}$ inches long and $\frac{5}{16}$ inch thick (§ 97 and § 111, Cooper), and they will be $15 \times \frac{5}{16} \times 21$ inches. The distance between gage lines is $8\frac{3}{4}+2\times 1\frac{3}{4}=12\frac{1}{4}$ inches, and $12.25\times\tan 60°=7.1$ inches, which is the maximum allowable spacing for single lattices. Seven-inch spacings will be used, thus making $3\frac{1}{2}$-inch spacing for the flats. The unsupported lengths of lattices is $(7^2+12.25^2)^{\frac{1}{2}}=14.1$ inches (§ 111, Cooper); the thickness must be $14.1/40=0.353$ inch $=\frac{3}{8}$ inch, and the width $2\frac{1}{4}$ inches. By § 110 of the Specifications the pitch of rivets in cover plate must not be greater than $4\times\frac{3}{4}$ inch $= 3$ inches for a length of $2\times 15=30$ inches. The pitch of the remainder should not exceed 5 inches. Care must be taken that the batten plates do not interfere with the lower chord and the hip vertical U_1L_1. The lower flanges of channels are cut back to allow U_1L_1 to be placed close to the web, and thus reduce bending stress on pin. The webs of the channels are cut back from the miter line $\frac{1}{8}$ inch, thus allowing a $\frac{1}{4}$-inch clearance for rotation around the pin. The top plate at the hip takes no stress, it being simply to prevent lateral motion and also to prevent dust and water from entering the joint. To give room for the inner foot-

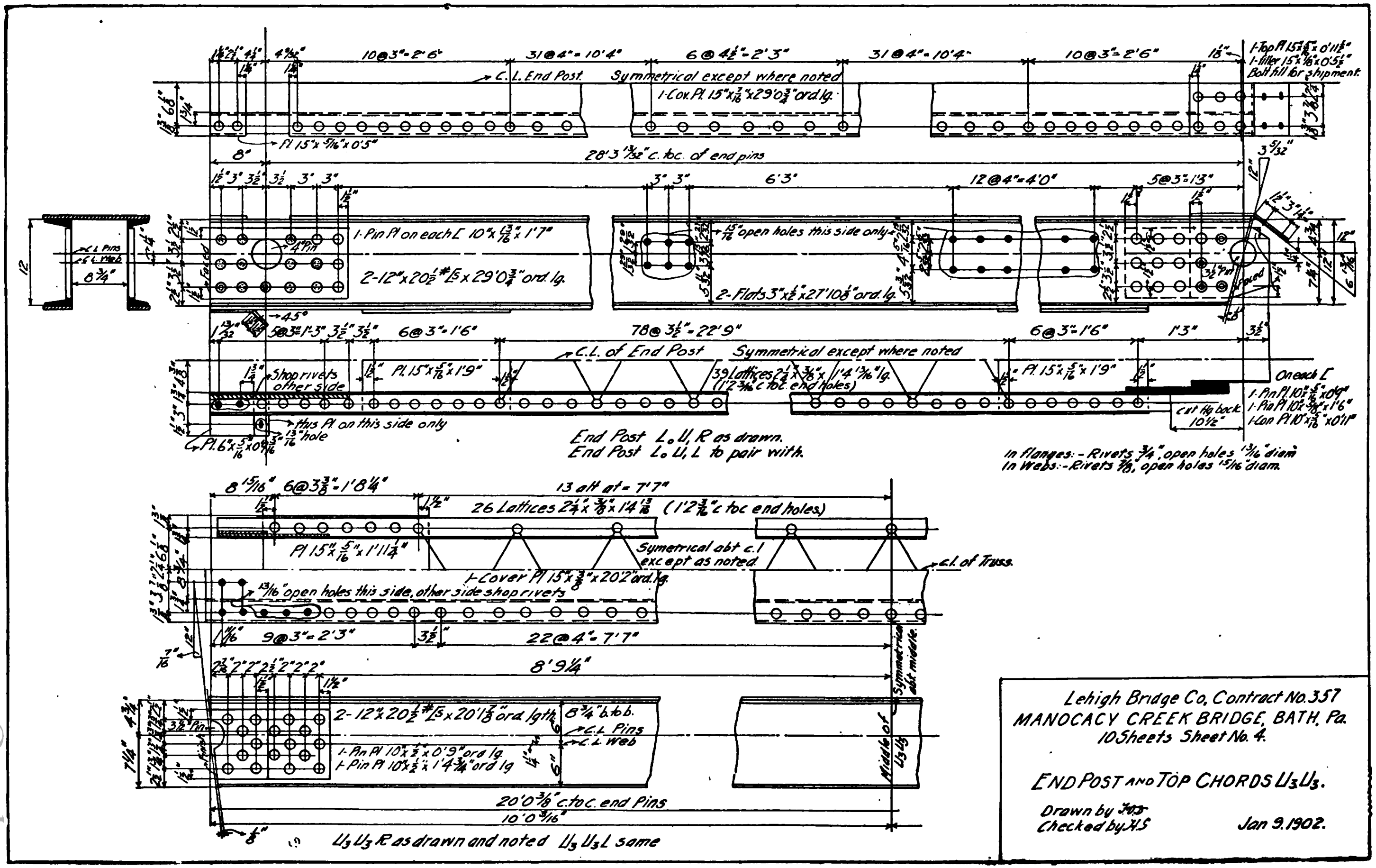

Lehigh Bridge Co, Contract No. 357
MANOCACY CREEK BRIDGE, BATH, Pa.
10 Sheets Sheet No. 4.
END POST AND TOP CHORDS U3U3.
Drawn by
Checked by H.S
Jan 9. 1902.
Symmetrical except where noted
C.L. End Post
End Post L0 U1 R as drawn.
End Post L0 U1 L to pair with.
In flanges:- Rivets 3/4", open holes 13/16" diam.
In Webs:- Rivets 7/8", open holes 15/16" diam.
Symetrical abt c.l except as noted
c.l. of Truss.
U3 U3 R as drawn and noted U3 U3 L same

Fig. 141.

walk stringer connection, it was necessary to extend the lower end of the end post 8 inches beyond the center of pin.

The pin plates, together with the required number of rivets, will now be computed (see Chap. IX). The total stress is 152 100 pounds, which is transferred to the pin equally by the two channel webs. The total bearing area required on each side is 76 050/18 000 = 4.21 square inches, and the total thickness of bearing area on one side on a $3\frac{1}{2}$-inch pin is 4.21/3.50 = 1.21 inches. The channel web being 0.28 inch thick, the pin plates must be 1.21 − 0.28 = 0.93 inch thick. They will consist of one $\frac{5}{8}$-inch and one $\frac{5}{16}$-inch plate, both placed outside so as to allow U_1L_1 to fit closely against the side of the web and thus reduce the bending stress on the pin. An additional $\frac{5}{16}$-inch plate with full pin hole is placed on top of last plate to facilitate erection.

The $\frac{5}{8}$-inch pin plate carries $0.625 \times 3\frac{1}{2} \times 18\,000 = 39\,400$ pounds stress, the web $0.28 \times 3\frac{1}{2} \times 18\,000 = 17\,600$ pounds stress, and the $\frac{5}{16}$-inch pin plate $76\,050 - (39\,400 + 17\,600) = 19\,050$ pounds stress. The value of a $\frac{7}{8}$-inch shop rivet in bearing in a 0.28-inch plate is 4400 pounds, and the number of rivets needed in bearing in the 0.28-inch web is $(39\,400 + 19\,050)/4400 = 13.2$, or say 13, as it is within $2\frac{1}{2}$ percent variation (§ 162, Cooper). There are required 58 450/6010 = 10 rivets in shear to connect both plates to web, 58 450/9850 = 6 rivets in bearing in the $\frac{5}{8}$-inch plate, 19 050/4920 = 4 rivets in bearing in the $\frac{5}{16}$-inch plate, and 19 050/6010 = 4 rivets in shear to connect the $\frac{5}{16}$-inch to the $\frac{5}{8}$-inch plate. From this it will be seen that the joint will fail first in bearing in the web, and that 13 rivets are required, 4 of which must pass through the $\frac{5}{16}$-inch plate.

The pin plates for the lower end, or joint L_0, will next be computed. Here both plates are placed on the outer side of

the post in order to allow the vertical plates of the shoe to be packed as closely to the backs of the webs of the channels as possible. A 4-inch pin being used, it is found, in the same manner as before, that one $1\frac{3}{16}$-inch plate is needed on each channel. Thirteen rivets are needed in bearing in the channel web to make the connection safe. Rivets are countersunk and chipped, where necessary, in order to allow members to fit close up to the web. When the views of a member are symmetrical about an axis, or nearly so, it is usually the practice to draw only one-half of such views; this occurs here, one-half views being drawn and exceptions noted. The top chord section U_8U_8 being symmetrical about two axes is drawn as shown, and, in order to economize space, the one-quarter top views and longitudinal sections are placed together. Open holes are shown on top to take the connections of the transverse and the top lateral bracings and a top plate. By § 133 of Specifications $\frac{3}{8}$ inch must be added to the length from center to center of pins. In this case, a $3\frac{1}{2}$-inch pin being used at both ends, the pin plates will be the same. The maximum stress is $133\,100 + 29\,400 = 162\,500$ pounds, one-half of which is transmitted to the pin by each side of the post. Following the same method as employed in the end post, the bearing area needed on one side is 1.285 inches, and the thickness of the pin plates 1.005 inches. Two $\frac{1}{2}$-inch plates will be used, both on the outside, as the post will not allow clearance enough to be placed on the inside. Fifteen $\frac{7}{8}$-inch rivets are required in the channel web, 5 passing through the outer plate. One more than the required number is used in order to make spacing symmetrical. Rivets in the top cover plate are spaced according to § 66 and § 110 of the Specifications. It will be noticed that, in the pin plates, the first row of rivets next to the edge of the smaller plate is closer than $1\frac{1}{4}$ inches, the distance required by the shop in order to drive a rivet. This is not, however, a violation of

shop practice, as that row of rivets can be driven before the smaller plate is put on, thus saving material in the length of the pin plate.

SHEET 5. TOP CHORD SECTIONS U_1U_2 AND U_2U_3. — The detailing of these presents no new features. Camber, rivet spacing, batten plates, and lattices are the same as before. Plates with full pin holes and top plates should be placed at the upper ends of each chord section. Open holes should be placed at the ends in order to take the connections of transverse and lateral bracings, and of top plates.

SHEET 6. PORTAL BRACING AND FOOT-WALK BRACKET. — As the portal bracing is symmetrical about its middle, only one half is required to be drawn. The connection plates are $\frac{5}{16}$ inch from back to back (Art. 116). At intermediate points rivets are spaced every 8 or 10 inches, and the angles are kept apart by two round washers $\frac{5}{8}$ inch thick and $2\frac{1}{4}$ inches in diameter. In the connection of the knee brace the rivet holes for the connection to the end posts and top chords U_1U_2 correspond. The maximum shear is 16 850 pounds (see Art. 116), which requires (§ 53, Cooper) 16 850/4600 = four $\frac{7}{8}$-inch field rivets in bearing in the $\frac{5}{16}$-inch angles connecting the portal strut to the end post. The stress in the knee brace is 23 700 pounds. Its vertical and horizontal components are 16 850 pounds. To connect the knee brace to the end post four $\frac{7}{8}$-inch field rivets are required in bearing in $\frac{5}{16}$-inch angles, and to connect it to the portal strut three $\frac{7}{8}$-inch shop rivets in bearing are needed in the $\frac{5}{16}$-inch plate. Four $\frac{7}{8}$-inch field rivets are required for bearing in the channel web.

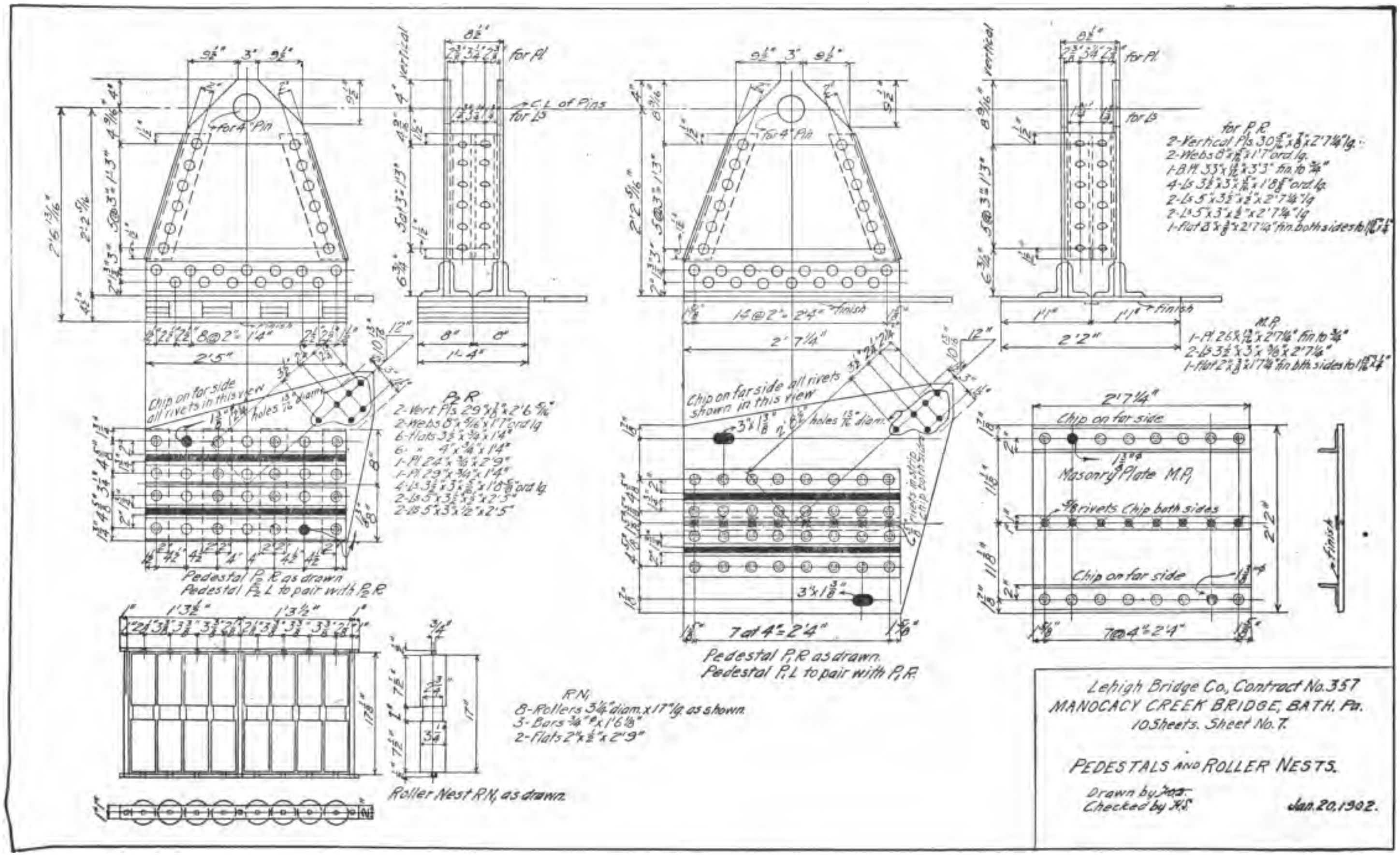
Each panel of the portal strut being about 4 feet, the length of a diagonal from center to center of gravity of flange angles is $(6^2 + 4^2)^{\frac{1}{2}}$, or about 7.2 feet, the secant of the angle which it makes with the vertical is $7.2/6 = 1.2$, and the stress in a diag-

onal is $1.2 \times 16\,850 = 20\,150$ pounds, the diagonals being considered as taking tension only. The number of $\frac{7}{8}$-inch shop rivets required for bearing in the ends is $20\,150/6900 = 3$. In detailing the diagonals the distance from center to center of end holes is given and the ordered length is billed on them.

The depth of the foot-walk bracket at its outer end is sufficient to allow a small clearance between flange angles. Plates $\frac{3}{8}$ inch thick are placed on the top flange to evenly distribute the reaction of stringers. Holes are placed in these corresponding to those in the ends of foot-walk stringers. The inner foot-walk stringer must clear the eye-bars and pins. The inner plate also takes the angles of the connection to the post. This plate has six $\frac{7}{8}$-inch shop rivets in it, in addition to the two field of the stringer connection. The strength of the joint in bearing in the $\frac{5}{16}$-inch angles is $(6 \times 3930 + 2 \times 2620) = 28\,820$ pounds, while the flange stress is only 23 700 pounds, thus showing it to be safe. The flange stress at the end is zero, the moment under the middle stringer is $4200 \times 2.5 \times 12 = 126\,000$ pound-inches, and taking the effective depth here as 11 inches, the flange stress is 11 400 pounds. The flange stress at the post, as previously computed, is 23 700 pounds. The difference of the flange stress between the end and the center is 11 400 pounds, requiring three $\frac{7}{8}$-inch shop rivets in bearing in the $\frac{5}{16}$-inch web. The difference in bending moments between the center and post is 12 300 pound-inches, requiring 4 rivets.

SHEET 7. PEDESTALS AND ROLLER NEST. — By § 130 of the Specifications, the vertical webs must be connected transversely when of sufficient height. This is done by means of angles and plates. The plates in this case should not be the full width of $8\frac{1}{2}$ inches, but less, in order not to interfere with the channels of the end post. The distance out to out of vertical plates is made $\frac{1}{4}$ inch less than the distance from back to back of channels for the same reason. The elevation of the under surface

of the bearing plate should be of the same elevation as that of the under surface of a $\frac{5}{16}$-inch lateral connection plate on the bottom of the floor beams. This is necessary so that the angles of the lower lateral system may be horizontal. The $3\frac{1}{2}$-inch vertical leg of these angles should clear the masonry. This requires that the distance from the under side of the bearing plate to the masonry shall be greater than $3\frac{1}{2}$ inches. The rollers at the roller end render this distance sufficient, but at the fixed end the pedestal must be built up by flats. The open holes in the bearing plates are for the lower lateral connection. The stress in the end lateral is 27 180 pounds, and (§ 53, Cooper) six $\frac{7}{8}$-inch field rivets are needed in bearing in the $\frac{5}{16}$-inch plate. The number of rivets in the vertical legs of connection angles must be sufficient to bear the entire reaction of 115 950 pounds, as no reliance can be placed in the bearing of plates. This joint is weak in double shear, requiring a total of fourteen $\frac{7}{8}$-inch shop rivets. It is thought best to space the rivets near the minimum spacing limit. This gives 15 required at the roller, and 13 required at the fixed end. The number in the horizontal leg cannot be computed except from the horizontal thrust of bridge. This gives a total required number of six $\frac{7}{8}$-inch rivets in single shear, friction being neglected. In all cases where rivets extend to a surface required to be flat, those heads are countersunk and chipped. The rollers (§ 127, Cooper) are of machinery steel. They should be ordered $3\frac{3}{8}$ inches in diameter and finished to the sizes given. The small flats riveted on the roller sides of masonry and bearing plates at the roller end prevent lateral motion to a slight extent and keep the rollers in alignment. They should be finished so as to give a $\frac{1}{16}$-inch clearance on all three sides between the roller and itself. The holes for anchor bolts (§ 131, Cooper) in both the masonry plates and in the bearing plate of the fixed end must be $1\frac{3}{8}$ inches in diameter. For the roller end those in bearing plate are oblong in order



Lehigh Bridge Co., Contract No. 357
MANOCACY CREEK BRIDGE, BATH, Pa.
10 Sheets. Sheet No. 7.
PEDESTALS AND ROLLER NESTS.
Drawn by
Checked by
Jan. 20, 1902.
Pedestal P2R as drawn
Pedestal P2L to pair with P2R
Pedestal P1R as drawn.
Pedestal P1L to pair with P1R
Roller Nest R.N. as drawn
Masonry Plate M.P.
Chip on far side
Chip on far side all rivets shown in this view
C.L. of Pins
for 4" Pin
finish

Fig. 142.

to allow for longitudinal motion. Their length is equal to the diameter of the anchor bolt plus the movement due to temperature.

SHEETS 8, 9, AND 10. TOP LATERALS, LOWER LATERALS, TRANSVERSE BRACING. — The detailing of these presents no new features over that of the portal strut. In all cases, the connection plates being $\frac{5}{16}$ inch thick, the joints will be weak in bearing in a $\frac{5}{16}$-inch plate. The stresses in the members are small, and rivets computed from these stresses do not give a sufficient number for good stiff details. Enough rivets to take up the entire strength of angles in tension should be used in all cases; when possible $\frac{7}{8}$-inch rivets should be used. Where two angles cross with backs in same plane and the legs on same side of the plane, as in lateral systems, one angle should continue, the other should be cut and a connecting plate used. This plate must be of sufficient cross-section to develop the full strength of angle.

ART. 119. ESTIMATE OF WEIGHT.

In practice the estimate of weight and cost is made long before the detailing is finished, in fact before the contract is let, and the exact weight is determined by weighing each member separately as it leaves the shop for shipment. For the student, the two corresponding operations are called approximate weight and computed weight. The former is made by formula or by comparing with some structure of like span and design, the latter by computing the weight, from the tables in handbooks, of each member, after it has been detailed, and, after multiplying by the number required, adding the results together, the sum being the computed weight of bridge. This sum is liable to an error of about one percent either way, due to inaccuracies of rolls in the mills. Each pair of $\frac{7}{8}$-inch rivet heads weighs about 0.45 pound, and each pair of $\frac{3}{4}$-inch rivet heads about 0.28 pound. The

following is an example of a convenient method of arranging the estimate of weight. It is for the intermediate posts of Sheet 3 (Fig. 140).

8 INTERMEDIATE POSTS. SHEET 3.					
Number.	Shape.	Size in inches.	Length.	Weight per linear foot in pounds.	Total weight, pounds.
8	Channels	10 inch × 20 pound	28 ft. $8\frac{3}{4}$ ins.	20.00	4 600
8	Channels	10 inch × 20 pound	27 ft. $2\frac{3}{4}$ ins.	20.00	4 373
16	Plates	$7 \times \frac{5}{16}$	1 ft. 8 ins.	7.44	199
32	Plates	$7 \times \frac{5}{16}$	1 ft. 0 ins.	7.44	238
8	Angles	$4 \times 3 \times \frac{3}{8}$	0 ft. 8 ins.	8.50	45
1408	Lattice Bars	$2 \times \frac{5}{16}$	0 ft. 8 ins.	2.65	2 486
				Total weight in pounds =	11 941

A similar table made out for the four hangers of Sheet 3 determines their weight to be 772 pounds. The rivet heads should now be counted and added to the above, thus giving the computed weight of all the intermediate posts and hangers. In like manner, the members and details on each of the sheets may be tabulated, and thus the entire weight of the bridge can be determined with a precision closely equal to that of actual weighing on scales.

CHAPTER XI.

RAILROAD RIVETED BRIDGES.

ART. 120. FORMS OF TRUSSES.

As stated in Art. 70, the lower limit of span for riveted trusses ranges from 75 to 100 feet, and the upper limit from 120 to 150 feet, or even to 200 feet, according to different specifications and standards.

The types of trusses in most general use for riveted bridges are the Warren with sub-verticals, the Pratt, and the Baltimore. For the shorter spans where deck trusses cannot be built on account of local conditions, pony or half-through bridges are used whose floor system and transverse bracing of the trusses is similar to that of through plate-girder bridges. In larger spans, under the same conditions, the trusses are connected by lateral, portal, and sway bracing as in pin-connected bridges. Partial detail drawings of a pony truss whose span is 120 feet are given in Art. 121, those of a 170-foot through Pratt truss are shown on Plate VI, Art. 122, while the standard details of through Baltimore trusses for spans from 100 to 200 feet are shown on Plate VII, Art. 123. Illustrations of a Warren truss with sub-verticals for a span of 114 feet 3¼ inches may be seen on the inset of Engineering News, July 9, 1896.

The double intersection Warren truss is also used to some extent, sub-verticals being added where the panels are subdivided in order to secure a shallow floor. The use of more than a single system of web members is not regarded with favor in the

best practice because the stresses are not statically determinate. In some instances the stresses in double intersection Warren trusses are made indeterminate to a still higher degree by the insertion of long verticals connecting the two systems. This is contrary to the line of progress described in Chap. I.

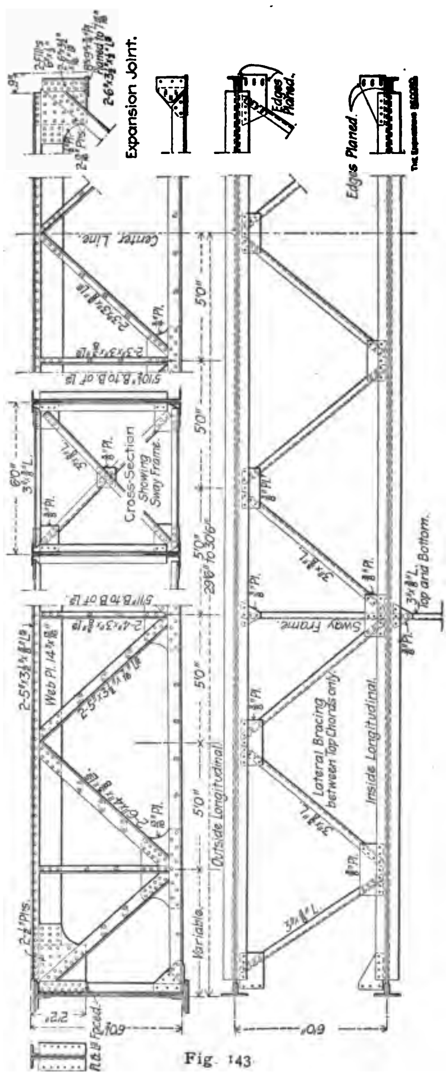


Fig. 143

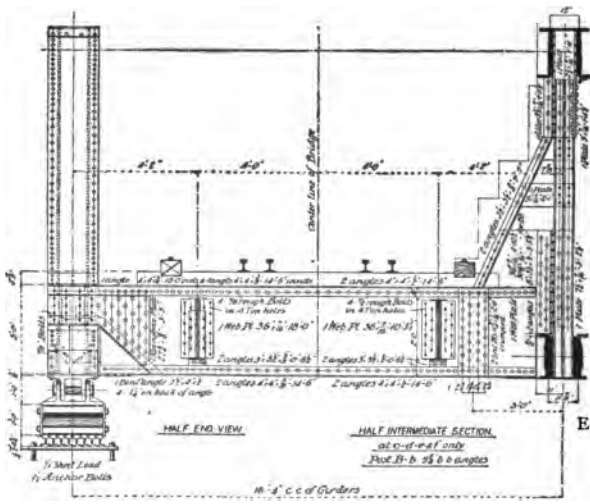
On elevated railroads, girders whose spans range from 40 to 65 feet are often required to be built with open webs in order to admit more light beneath the structure than the solid webs of plate girders. This requirement applies to most locations except those where the elevated structures occupy the middle of a very wide street. Fig. 143 shows the plan, elevation, and cross-section of a half span of the Boston Elevated Railroad. The deck trusses, whose depth is very nearly six feet, are of the Warren type with sub-verticals. The upper chord is subject to combined compression and flexure, and is composed of a pair of angles and

a web plate, while the lower chord and all of the web members consist only of pairs of angles, separated by washers whose thickness equals that of the connecting plates. The lateral and sway bracing is the same as that of a deck plate-girder bridge.

ART. 121. DETAILS OF A LATTICE GIRDER.

Figs. 144, 145, and 146 show most of the details of a pony truss bridge whose span is 120 feet, taken from a standard plan of the Northern Pacific Railway. On the standard plan it is designated as a through lattice girder, and this term is very generally employed in practice, although strictly the term lattice girder applies to one with two or more systems of webbing.

Fig. 144 shows that the intermediate floor beams are connected to the sub-verticals above the lower chord. The end



Eng. News.

Fig. 144.

web plates do not extend to the top of the inclined stiffening angles, but only as high as the end connecting angles. The web splice plates are not limited to the clear space between the

flange angles, but are crimped over the vertical legs of these angles. The end floor beam is riveted by means of angles to the large plates connecting the corresponding sides of the end post and the lower chord. The lower flange angles continue straight to the lower end of the web plate, instead of being bent upward and extended over the end post, as on Plate IV. The lower flanges of all floor beams are riveted to the connecting plates of the lateral system. The trusses are spaced 16′ 4″ center to center, and the clear distance between the upper chords is 14′ 6″. Each stringer has a web plate $24'' \times \frac{7}{16}''$, and flanges composed of two angles $5'' \times 3'' \times \frac{3}{8}''$. The stringers have no separate lateral system, but each one is connected to both of the laterals of the bridge by means of short angles. Brackets in line with the stringers project 15 inches beyond the end floor beams, and support one cross-tie at each end of the span.

The sub-verticals are composed of two pairs of angles united by four web or tie plates, two of which are extended inward to connect with the inclined stiffening angles, as shown in Fig. 144. The diagonals *Bc* and *cD* (Fig. 145) consist of two pairs of unequal-legged angles united by a continuous web plate, while in the diagonals *De* and *eF* the angles are connected by four tie plates, the intermediate ones being much smaller than the end ones (Fig. 146). The upper chord is composed of two web plates, a cover plate, and four angles, the lower angles being larger than the upper ones. The increased chord section from *D* to *F* is provided by means of two additional web plates. The lower angles of the chord are united by tie plates and single lacing. The composition of the end post is the same as that of the chord *BD*. In the lower chord the angles have the same size throughout. From *a* to *c* the two web plates are only $\frac{3}{8}$ inch thick, from *c* to *f* there are four web plates $\frac{11}{16}$ inch thick, and from *e* to *f* two side plates are added in the clear

space between the flange angles. The chord is laced on both top and bottom.

The joint or connecting plates are $\frac{3}{4}$ inch thick and are riveted to the inside of the webs of the chords and to the backs of the angles of the web members. Fillers are inserted at the

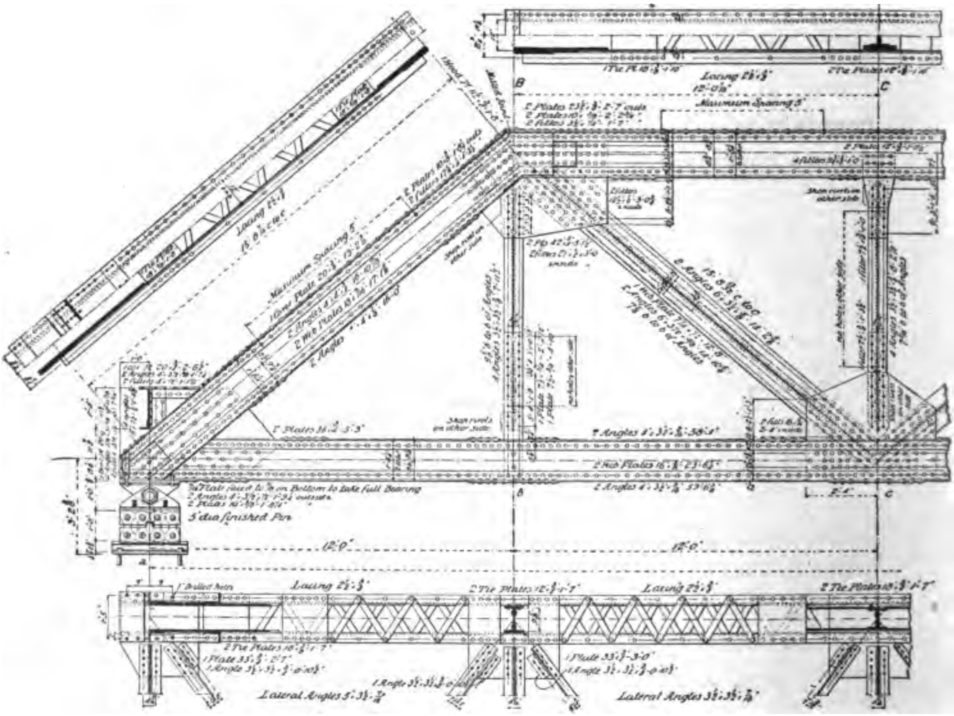


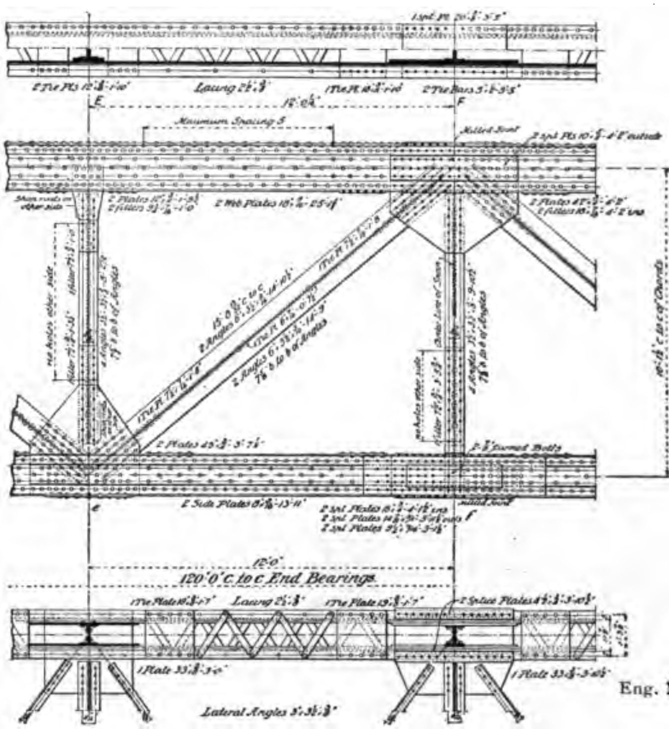
Fig. 145

upper joints on account of the difference in the inside clear spacing of the two chords.

The truss is shipped in two parts and spliced in the field as indicated in Fig. 146. The connecting plates act as splice plates, and in addition two side plates and one cover plate are provided for the splice. At *B* two side plates, extending over

the vertical legs of the flange angles, as well as two fillers, are added as splice plates. The joints at both B and F are milled. The details of the end bearings are similar to those for plate girders. See the standard plan, Plate II.

This girder was designed for the same standard loading as



Eng. News

Fig. 146.

that described in Art. 102, and the weight of a complete span of the bridge is about 215 000 pounds. A riveted through span with overhead bracing was also designed for a span of 110 feet, and its weight was found to be 19 000 pounds less than that of the lattice girder of the same span; but since the former required more field-riveting than the latter, and as the absence

of overhead bracing was regarded as a desirable feature, the girder span was adopted as the standard. See Journal Western Society of Engineers, vol. 6, page 53, Feb. 1901.

Joints and connections in riveted work, whether in tension or compression, are designed to develop the full strength of the members, proper provision being made for field riveting. The connecting plates must have a thickness proportioned to the amount of stress to be transferred, and must properly distribute the web stresses to the plates and shapes which compose the chords. The 1901 specifications of the Baltimore and Ohio Railroad limit the number of rivets in any connection of a $\frac{3}{8}$-inch plate to ten.

When but few lines of rivets are used in any connection and the lines are long, the elongation of the member within the limits of the connection makes a very unequal distribution of the stress to the rivets. This consideration will often determine the question whether to employ angles with legs wide enough to permit two rows of rivets instead of one.

Where a number of rows of rivets are inserted in tension members, it is important to make a sufficient deduction for rivet holes. See Experiments on Iron and Steel Joints Riveted on Angle by B. B. FLINT in Transactions of American Society of Civil Engineers, vol. 27, page 406, Oct., 1892. See also Net Section in Riveted Work by THEODORE COOPER in Railroad Gazette, vol. 22, page 583, Aug. 22, 1890.

ART. 122. DETAILS OF A PRATT TRUSS.

Plate VI contains parts of a general drawing of a riveted Pratt truss whose span is 170 feet. It was taken from one of a series of standard plans of riveted bridges having a considerable range of span, and which were designed for class W of WADDELL's compromise standard live loads (Art. 32). All

the material is medium steel except the rivets and anchor bolts, which are of soft steel. The trusses are spaced 17 feet apart between centers, while the stringers are spaced 7 feet apart.

The stringers (not shown on Plate VI) have web plates $39\frac{3}{4}'' \times \frac{3}{8}''$, and flanges of two angles $6'' \times 4'' \times \frac{1}{2}''$, the long legs being horizontal. There are seven intermediate pairs of stiffeners crimped over the flange angles. The end connecting angles have fillers under them twice as wide as the angles. The lateral system of the stringers in each panel consists of four diagonals, each composed of one $3\frac{1}{2}'' \times 3\frac{1}{2}'' \times \frac{3}{8}''$ angle. The stringers are connected by means of two short angles to the lower laterals of the bridge in a very effective manner in accordance with the specification given in Art. 94. The connection of one of the transverse braces between the lower flanges of the stringers is shown on the plate.

The floor beams have intermediate stiffeners. Their connection to the posts and suspender is very simple, since these members have the same width as the stiff lower chord. The web of the end floor beam is reënforced at each end by a plate 25 inches wide inside of the end connections. A diaphragm like that in the verticals is placed between the large connecting plates at the panel point L_0.

The posts and suspender have sections like those for pin-connected trusses. The diagonal U_1L_2 consists of two plates and four angles united by a single line of lacing, while the diagonal U_2L_3 has two channels with the flanges turned inward and united by two lines of lacing. In the middle panel there are two stiff diagonals, each composed of two pairs of angles connected by a single line of lacing. Both diagonals are cut at their intersection and riveted to a pair of connecting plates.

The upper chord and end post consist of a cover plate and two channels connected below by tie plates and single lacing.

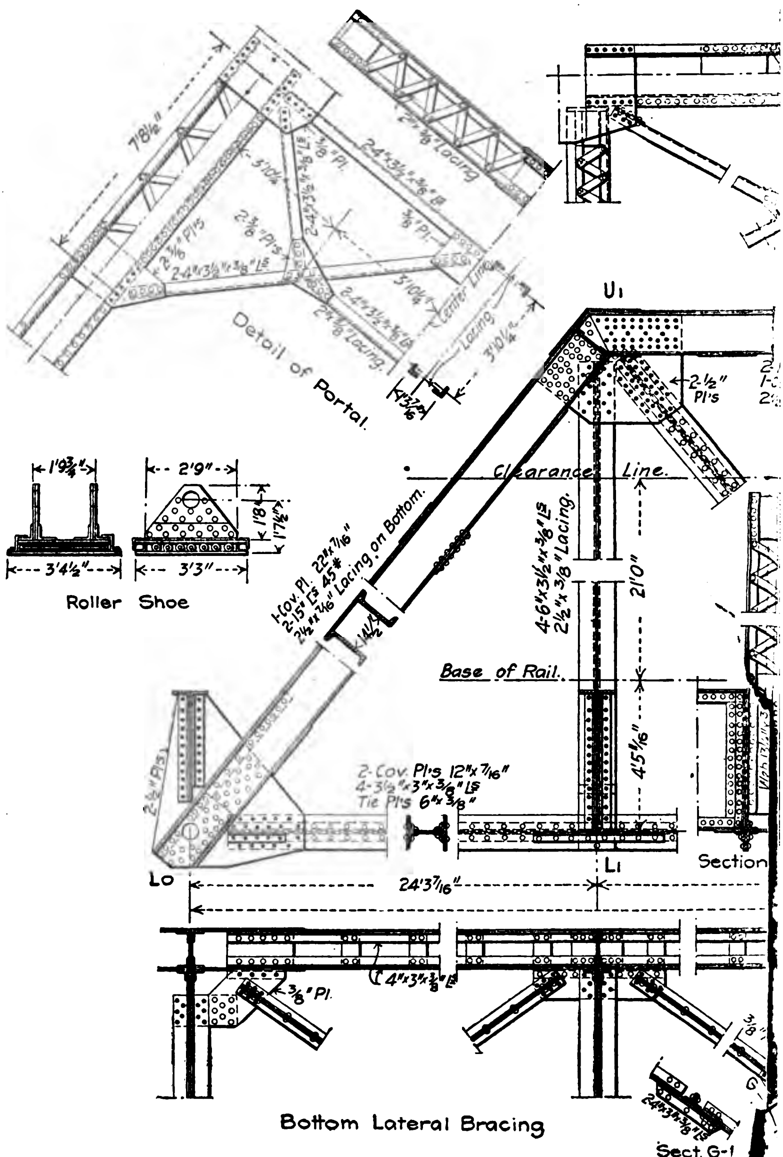
The splices of the upper chord are similar to those of a pin-connected truss. From L_0 to L_2 the lower chord section is composed of two web plates and four angles united by a series of narrow tie or batten plates, as shown on the drawing. From L_2 to L_3 two plates of the same section are added and the angles increased to $4'' \times 3'' \times \frac{7}{16}''$, while in the middle panel the chord consists of two plates $12'' \times \frac{9}{16}''$, two plates $12'' \times \frac{5}{8}''$, and four angles $4'' \times 3'' \times \frac{7}{16}''$. The entire chord is spliced on the left of L_2, the composition of the splice being given on the plate. There is a similar splice also at the left of L_3.

The upper laterals are made up of two angles $4'' \times 3'' \times \frac{3}{8}''$, laced together so as to form a stiff member as deep as the chord, and attached by connecting plates to the top and bottom of the lateral struts as well as to the chords. Each of the lower laterals consists of two angles $4'' \times 3'' \times \frac{3}{8}''$, placed with the $4''$ leg vertically and riveted together every foot. The splice at the intersection of the laterals has two angles of the same size in addition to the $12'' \times \frac{3}{8}''$ plate in order to give stiffness as well as strength to the splice. The end connecting plates are riveted to the bottom flanges of the floor beams and to the shelf angles attached to the side of the chords.

The portal bracing consists of two small trusses, one of them connected to the upper and the other to the lower side of the end posts. All the corresponding members of these trusses are laced together in pairs, thus making a portal of considerable stiffness in all directions. The construction of the intermediate sway bracing is fully shown on the drawing.

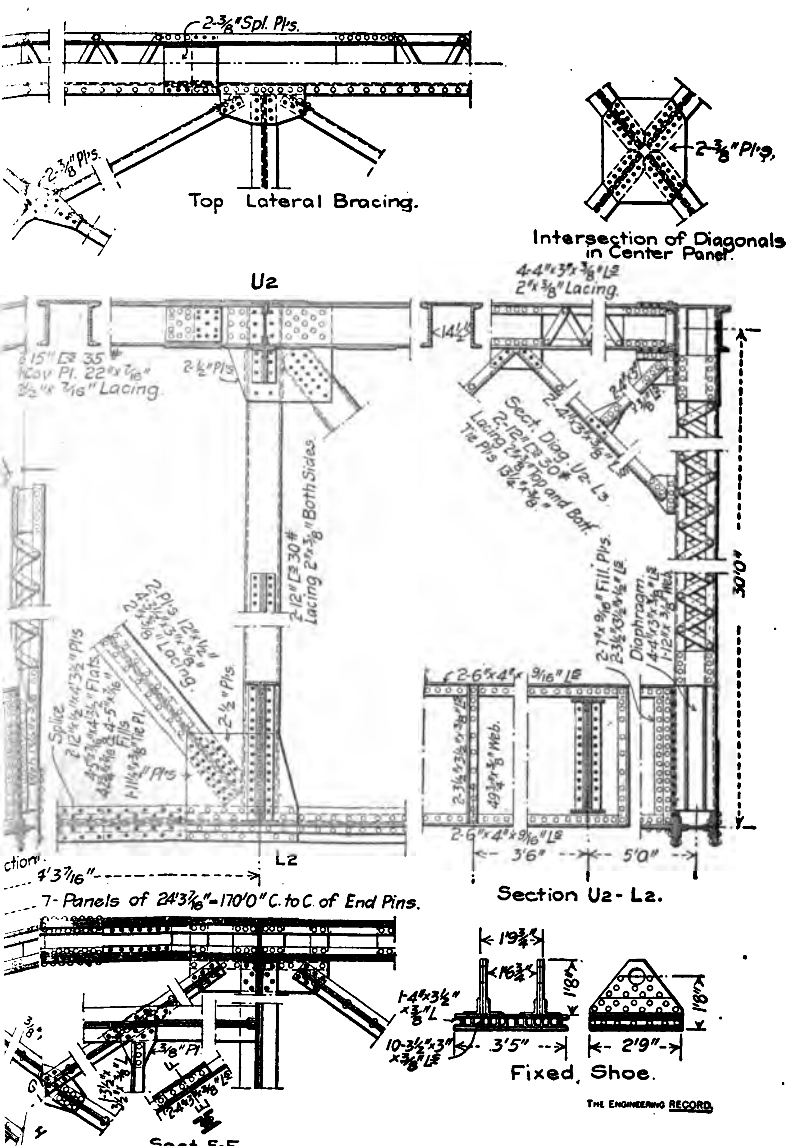
The connecting plates at the different panel points are all $\frac{1}{2}$ inch in thickness. They are riveted to the inside of the upper chord and end posts, and to the outside of all the other members. The reaction of the panel point L_0 is transferred from the end post to the pedestal by a 6-inch pin, the necessary bearing

Single-track Through Riveted T



PARTIAL DETAIL DRAWING OF ONE OF THE RIVETE
FOR THE VERA CRUZ A

l Truss Bridge. Span, 170 Feet.



RIDGES DESIGNED BY WADDELL AND HEDRICK IN 1899
ACIFIC RAILWAY IN MEXICO.

of the end post being secured by means of $\frac{5}{8}$-inch pin plates in addition to the large connecting plates. On account of the eccentric location of the pin, two angles are inserted to reduce the effect of this eccentricity on the pin plates and to aid in transferring its share of the stress into the cover plate.

The general construction of the pedestals or shoes at both ends is indicated on the drawing. The rollers are 3 inches in diameter. In the first shoe the $3\frac{1}{2}$-inch vertical legs of the angles are planed to 3 inches. The web or pin plates of the pedestals as well as the bearing and bed plates are $\frac{3}{4}$ inch thick, while the connecting angles are $6'' \times 6'' \times \frac{3}{4}''$. The anchor bolts are of soft steel $1\frac{1}{4}$ inches in diameter and 2 feet long, having cold-pressed threads and foxed ends.

The ends of stringers and floor beams are faced as well as those of abutting compression members in order to secure perfect contact. All rivet holes are punched $\frac{1}{8}$ inch less and reamed to $\frac{1}{16}$ inch greater diameter than that of the rivet. All truss members are assembled in the shop and the field-rivet holes reamed to a perfect fit.

In the shorter spans the diagonals U_1L_2 are made of two channels laced together instead of two pairs of angles similarly connected, and the sections of the end post are balanced by riveting flats to the lower flanges of the channels.

A riveted truss bridge of the same type whose span is 150 feet is described and illustrated in Engineering News, vol. 40, page 114, Aug. 25, 1898. It was designed by the same engineers for the Kansas City, Pittsburg and Gulf Railroad. Two of the principal differences in the details consist in the upper chord being laced on both the upper and lower sides, and in the portal bracing being a simple lattice girder, combined with knee braces, attached to the upper side of the end post. See also two communications on the comparative economy for short

spans of riveted trusses and the "A" type of pin-connected trusses, on page 346 of the same volume.

ART. 123. DETAILS OF A BALTIMORE TRUSS.

Plate VII shows the standard details for riveted bridges adopted by the New York Central and Hudson River Railroad. One of the reasons for adopting the Baltimore truss for riveted bridges was the necessity, in a number of instances, of shallow, solid trough floors requiring short panels. A large number of these trusses have been built during the recent extensive renewals on the principal lines of this railroad, involving in the aggregate, up to 1902, about 70 000 tons of steel bridges.

The plate shows the details of the connection of rectangular trough floors to the lower chord whose depth is greater than that of the upper chord, since it is subject to combined flexure and tension. When the floor system consists of stringers and floor beams, the bottom flanges of the floor beams are almost even with the bottom of the lower chord, and in order to secure adequate connections to the verticals of the trusses the web is spliced near each end, and the end web plates extended upward as a gusset plate or knee brace, in the same manner as for through plate girders (see Fig. 56). The splice plates extend beyond the ends of the outer stringers, thus serving also as filler plates, and these, together with the web plate, are slotted over the upper flange angles of the chord.

The short suspenders and the short diagonals, as well as the long sub-verticals in the smaller spans, are made up of two pairs of angles laced together. The long suspenders either have two plates in addition to the four angles, or they consist of two channels laced on both sides. The long diagonals and the lower chord consist of built-up channels with additional

web plates on the inside, or filler plates between the angles on the outside, in order to enlarge the section when needed. The sections of the upper chord and end post are similar to those which are built up of plates and angles for pin-connected trusses of short spans.

Both the portal and intermediate sway bracing consist of lattice girders with multiple systems of webbing, the flanges being composed of two angles and a plate which is wide enough for the connections of the web bracing. Brackets are also used, those of the portal having a solid web while the others have only a single pair of angles to stiffen the main knee-brace angles. The upper laterals are composed of two pairs of angles laced together so as to form a member as deep as the chords to which they are connected. The lower laterals consist of two angles riveted back to back, and are spliced to a common connecting plate which is riveted to the bottom flange at the middle of alternate floor beams. By means of angle clips they are also connected to the lower flanges of the stringers. The end connecting plates are attached to the bottom angles of the lower chords.

The lower ends of the trusses are connected by end floor beams, and are supported by pedestals whose pin bearings are located below the lower chord. The web or pin plates of the pedestals are stiffened by outside angles and by a connecting diaphragm, the pin taking bearing throughout its length. Alternative details are given for nests of cylindrical and segmental rollers respectively.

Figs. 147–149 give three views of one of the fixed spans of the New York Central and Hudson River Railroad passenger bridge at Albany, N. Y. The span is $181\frac{1}{2}$ feet long. The trusses are $30\frac{1}{2}$ feet deep and are spaced 29 feet center to center. The composition of the principal members together with a small-

scale illustration showing the character of the details, which conform to the standards described above, are given in Engineering Record, vol. 40, page 498, Oct. 28, 1899. The railroad improvements are also briefly described in Railroad Gazette, vol. 31, page 774, Nov. 10, 1899, and several views are given of the old bridge trusses as well as of the new ones which replaced them. The new structure was completed in 1900.

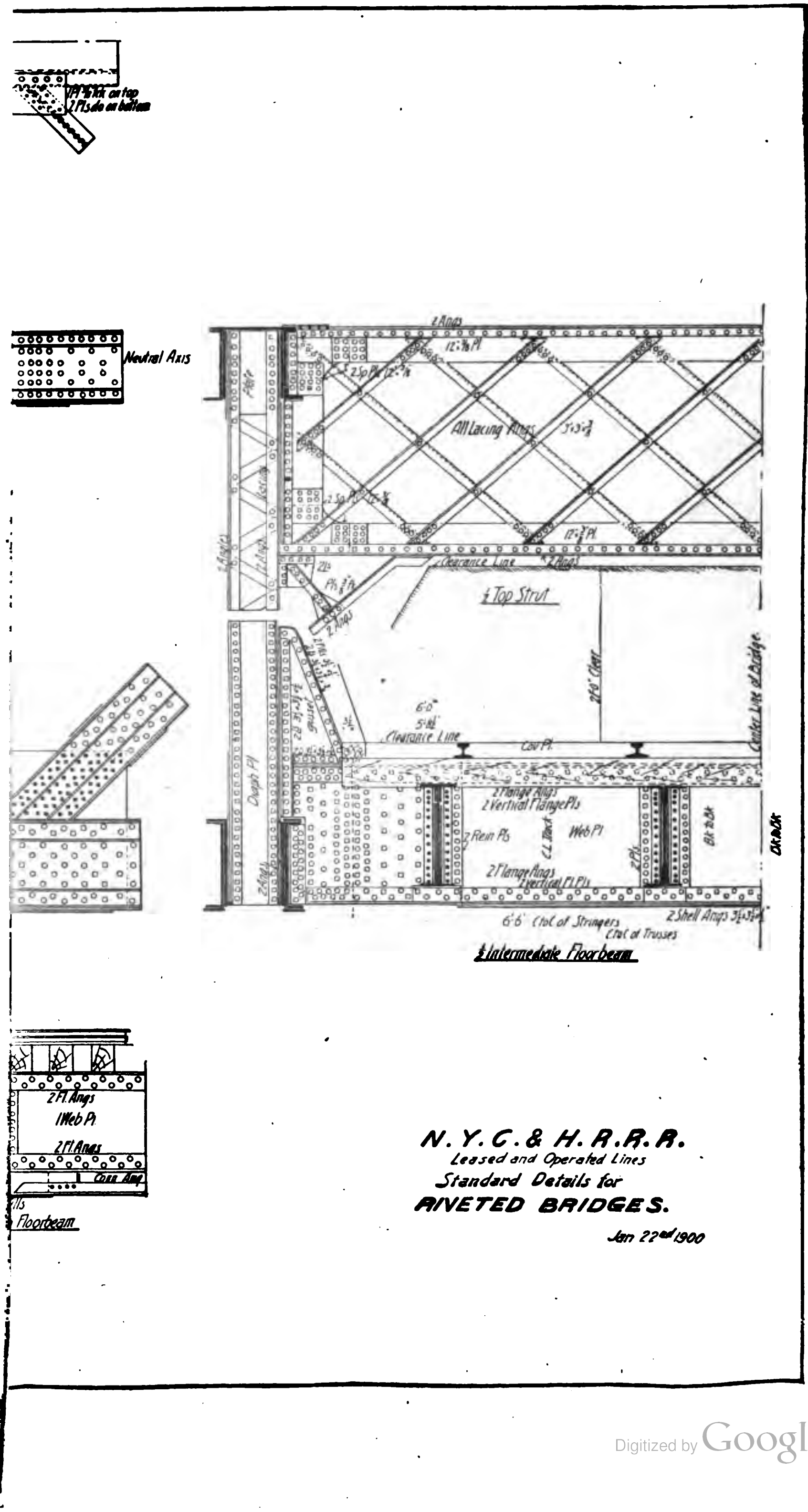
The views show clearly the connections at the joints, and the relation of the members meeting at the different joints, as well

Fig. 147.

as the forms of the members and their details. The bridge was designed for very heavy passenger service, and especial attention was given to securing stiffness as well as adequate strength in the truss members and in each span as a whole. These three illustrations, Figs. 147–149, are from photographs taken in January, 1902, and kindly furnished for publication here by W. J. Wilgus, Chief Engineer of the New York Central and Hudson River Railroad.

Fig. 148.

Fig. 149. Portal of Passenger Bridge, Albany, N. Y.



1 Pl. 7/16 thk on top
2 Pls. do on bottom
Neutral Axis
2 Angs
12"×3/8 Pl
2 Sp Pls 12"×3/8
All Lacing Angs 3½×3×3/8
12"×3/8 Pl
Clearance Line
2 Angs
½ Top Strut
Plate
Lacing
2 Angs
Diaph Pl
2 Angs
6'0"
5'6½"
Clearance Line
Cov Pl
2 Flange Angs
2 Vertical Flange Pls
2 Rein Pls
Web Pl
2 Flange Angs
2 Vertical Fl Pls
2 Pls
Center Line of Bridge
6'6" C to C of Stringers
C to C of Trusses
2 Shell Angs
½ Intermediate Floorbeam
2 Fl. Angs
1 Web Pl
2 Fl. Angs
Floorbeam
N.Y.C. & H.R.R.R.
Leased and Operated Lines
Standard Details for
RIVETED BRIDGES.
Jan 22nd 1900

INDEX.

A TEXT-BOOK ON ROOFS AND BRIDGES.

BY

PROFESSORS MERRIMAN AND JACOBY.

PART I.

STRESSES IN SIMPLE TRUSSES.

SIXTH EDITION, OCTAVO, CLOTH, PRICE $2.50.

CONTENTS:

Chap. I.—STRESSES IN ROOF TRUSSES.
II.—BRIDGE TRUSSES UNDER DEAD LOADS.
III.—BRIDGE TRUSSES UNDER LIVE LOADS.
IV.—FINAL STRESSES FOR BRIDGE TRUSSES.
V.—AMERICAN BRIDGE TRUSSES.
VI.—BRIDGE BRACING, MEMBERS AND FLOORS.
VII.—DEFLECTION AND LEAST WORK.
VIII.—MISCELLANEOUS STRUCTURES.

This edition has been rewritten and reset. Many new articles have been added, while the matter and arrangement of those retained have been materially changed. Nearly all of the examples and problems are new and in most cases relate to actual structures. Nearly all the illustrations are new. Numerous references to engineering periodicals have been added, while the historical notes have been largely extended. Compared with the fifth edition the number of chapters has been increased from six to eight, and the number of pages from 191 to 326; the number of line cuts has been considerably increased and thirty half-tone illustrations introduced, nearly all of which show bridges of modern design, and two folding plates have been added.

PUBLISHED BY

JOHN WILEY & SONS, 43 & 45 EAST NINETEENTH ST., NEW YORK.
CHAPMAN & HALL, LIMITED, LONDON.

A TEXT-BOOK ON ROOFS AND BRIDGES.

BY

PROFESSORS MERRIMAN AND JACOBY.

PART II.

GRAPHIC STATICS.

THIRD EDITION, OCTAVO, CLOTH, PRICE $2.50.

CONTENTS:

This edition contains nearly double the matter of former ones, and also two new plates. The determination of the positions of locomotive wheel loads which produce maximum stresses, has been reduced to the same simple operation of stretching a thread over a load line for trusses with curved chords as for those with horizontal chords. In order to reduce the analysis of stresses to its simplest form and to economize labor in the practical application, a special effort has been made to use such combinations of improved graphic methods as are best adapted to modern types of trusses, to arrange the tables in convenient form, and occasionally to replace intermediate graphic operations by computation.

PUBLISHED BY

JOHN WILEY & SONS, 43 & 45 EAST NINETEENTH ST., NEW YORK.
CHAPMAN & HALL, LIMITED, LONDON.

COPIES FORWARDED POSTPAID ON RECEIPT OF THE PRICE.